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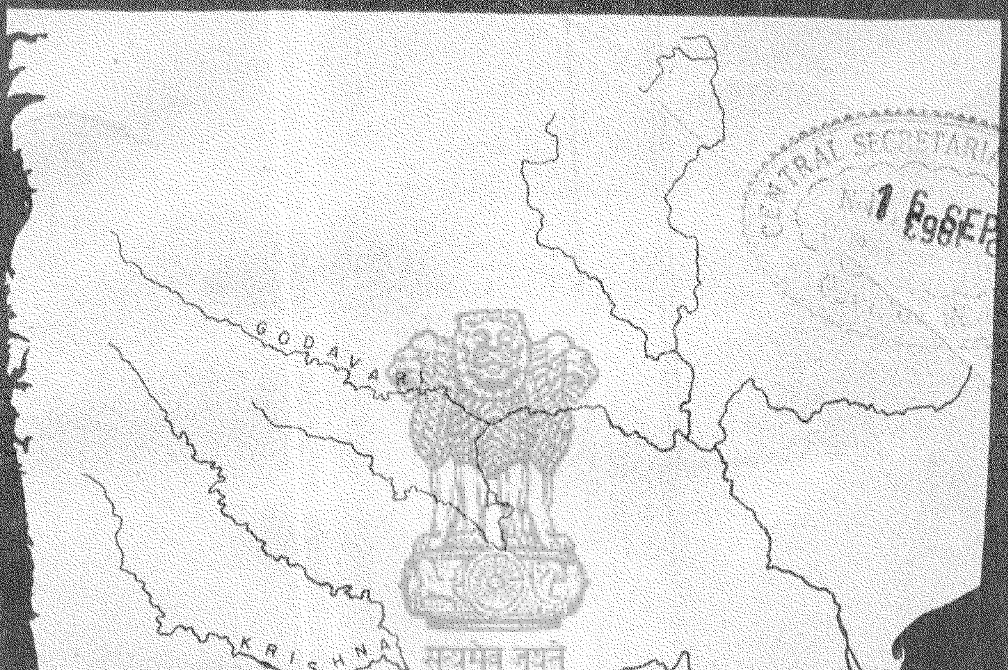
REPORT

OF

THE KRISHNA GODAVARI COMMISSION

MINISTRY OF IRRIGATION AND POWER



CENTRAL SECRETARIAT
16 SEP 1963

सत्यमेव जयते

ANNEXURE XVII

Model Experiments Regarding Flow over the Krishna Anicut at Vijayawada and the Godavari Anicut at Dowlaishwarm

Foreword

Some model experiments regarding flow over the Krishna Anicut had been carried out in 1949 at the Poondi Research Station, Madras ; the conclusions arrived at from these experiments are set out in this Annexure.

At the Commission's request, a fresh series of experiments were carried out at the Central Water and Power Research Station, Poona, and the Indian Institute of Science, Bangalore, regarding flow over the Krishna Anicut. The reports on these experiments are given hereunder. The Government of Andhra Pradesh also carried out some experiments in the same connection ; the report on these investigations is also included in this Annexure.

For flow conditions over the Godavari Anicut at Dowlaishwaram, some experiments were carried out, at the Commission's request, by the Central Water and Power Research Station, Poona. The report on these experiments is presented in this Annexure.

This Annexure also contains a comparative study of the recorded flow at Krishna Anicut and the flow as calculated on the basis of the results of experiment carried out by the Government of Andhra Pradesh.

सत्यमेव जयते

NOTATION

Unless otherwise stated the following notation has been adopted in this Annexure

D = Discharge in cusecs

L = Length, in feet, of weir or vent over which flow takes place

B = Width, in feet, of the weir (*i.e.*, the length along the direction of flow)

H = Depth of flow (upstream), in feet, above the crest of the weir

P = Depth, in feet, of upstream bed below the crest of the weir

V = Velocity in feet per second

Va = Velocity of approach in feet per second

C, C_1, C_2, C_3, Cd = Coefficients of discharge

R = Hydraulic mean radius

Lr = Linear scale of the model

S = Upstream slope

Sd = Downstream slope

$N(m)$ = Manning's rugosity cœfficient

$R.L.$ = Reduced level

d = Difference, in feet, between the water level downstream of a weir and crest of the weir

dc = d at the modular limit

h = Difference, in feet, of water levels above and below the weir

ha = Head, in feet, due to velocity of approach

g = Acceleration due to gravity in feet per second per second

q, q_1 = Discharge per foot length in cusecs

α = Coefficient

$\varkappa$ = Reduction factor

Vs - Versus

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सत्यमेव जयते

PART I

KRISHNA ANICUT AT VIJAYAWADA

Experiments by the Irrigation Research Station, Poondi, Madras

The following conclusions* were drawn from experiments conducted on a sectional model, to scale 1/48 of the Krishna Anicut at Vijayawada :

"(*i*) The Krishna Anicut functions in the free discharge condition for all depths of overflow obtained in the prototype notwithstanding the fact that for certain stages of flow the tail-water elevation is higher than the crest level.

"(*ii*) The discharge could be computed accurately making use of the equation $Q=CLH^{3/2}$ and the graph (not printed) connecting C and H for depths of overflow from 6 feet to 25 feet.

"(*iii*) For depths of overflow less than 6 feet the usual formula as per M.D.S.S.** may be followed.

"(*iv*) The percentage error committed by using the standard submerged weir formula increases with increase in head upto about 18 feet and it decreases with increase in head for higher values."

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* *Vide page* 5, *Madras Irrigation Research Publication No.* 6, *Annual Report for* 1949, *Public Works Department, Irrigation Research Station, Poondi, Madras.*

***Madras Detailed Standard Specifications.*

PART II

KRISHNA ANICUT AT VIJAYAWADA

*Experiments by the Central Water and Power Research Station, Poona**

1. *Terms of reference* :

The Chairman, Krishna Godavari Commission in his D.O. letter No. KGC-11/61 dated 5th January, 1962 enunciated the problem in the following terms. It was already agreed that all these studies would be restricted to a sectional two dimensional model of the Krishna Anicut in a flume :—

(*i*) Determine by model experiments, the modular range for the section of the Krishna anicut (as it existed before the construction of the Krishna Barrage) for all values of head over crest up to 24 feet ; in other words ascertain for each values of H (2 feet, 4 feet, 6 feet,...up to 24 feet), the highest downstream water level at which the flow over the Anicut is not affected, in any way, by the downstream water level.

(*ii*) Within the modular range and for different values of H determine (*a*) the value of coefficient C in the equation $q = CH^{3/2}$ and (*b*) the value of coefficient C_1 in the equation $q = C_1 (H+ha)^{3/2}$.

(*iii*) In determining the coefficients of discharge of C and C_1 it may be stated whether there is any scale effect and whether there is any effect of the level of the upstream bed on the value of the coefficients of discharge.

(*iv*) Verify by model experiments, the accuracy of the following equation:

$$Q = 3.1L[(h+h_a)^{3/2} - h_a^{3/2}] + C_2.L.d\ \sqrt{2.g(h+h_a)}.$$

2. *The experimental set up* :

Geometrically similar sectional models were constructed in a glass sided flume 4 feet wide. Since, during high floods the crest gates will be down, the crest level of the anicut (top shutters lowered down) would be R.L.47.22. This level was properly reproduced in all the models along with the slopes on both the sides as shown in the Anicut drawing *vide* Plate I. In all these models, the bed level on the downstream side was the same as at the end of the loose stone apron, *i.e.* R.L. 31·24.

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* *C.W.P.R.S. Specific Note No.* 544, *dated* 9*th April,* 1962.

For ascertaining the scale effect, two models were constructed :

(*a*) 1/40 scale geometrically similar sectional model with mobile bed ;

(*b*) 1/25 scale geometrically similar sectional model with mobile bed.

In both the above models the bed level upstream of the anicut was the same as at the end of the 14 feet long sloping apron *i.e.*, R.L. 39·55.

For ascertaining the effect of the upstream bed level on the coefficient of discharge two more models were constructed as below :

(*c*) 1/40 scale geometrically similar sectional model with upstream bed made rigid and laid at level 20 feet below the crest *i.e.*, at R.L. 27·22; and

(*d*) 1/40 scale geometrically similar sectional model with rigid bed on the upstream side laid 30 feet below the crest *i.e.*, at R.L. 17·22.

The gauges in all the above sectional models were located according to the standard laboratory practice. The gauge upstream of the anicut was located at a point equivalent to 96 feet away from the anicut, which was four times the maximum depth of flow expected over the same.

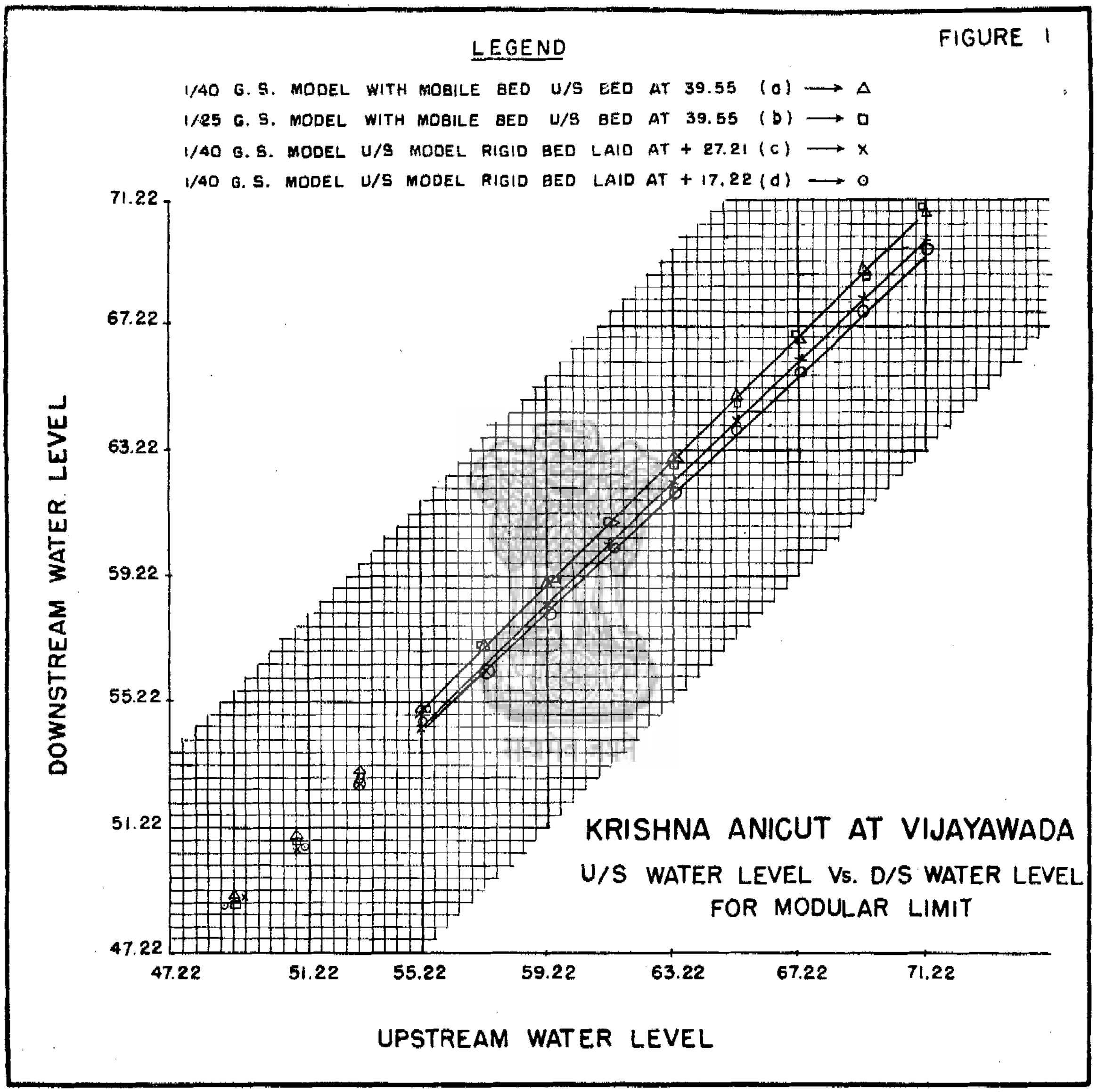
The downstream gauge was so located as to be free from turbulence below the hydraulic jump. At maximum depth of flow of 24 feet over the anicut, the hydraulic jump was seen to form about 52 feet below the weir crest. The height of this wave above the bed level was approximately computed at 30 feet . The downstream gauge was, therefore, located at 10 times the depth of the standing wave *i.e.* 10 × 30 feet from the position of the standing wave *i.e.* 352 feet below the crest of the anicut.

The above positions of the gauges remained unaltered for all the four models.

3. *The experiments* :

The experiments were conducted with the static head over the crest as measured at the upstream gauge at intervals of 2 feet from 2 feet to 24 feet. Discharge required was admitted to the model so as to attain the desired water level upstream. The water level on the downstream was maintained so as to form a clear standing wave on the downstream glacis slope. The downstream water level was then gradually raised by a control on the downstream till a limit was reached when any further raising of the downstream water level was found to affect the upstream water level also. This gave the limit of modular range. Photos 2281/61 and 2280/61 (not printed) show the conditions on either side of the modular limit with a static head on the crest of 24 feet as observed on the model (*a*) in paragraph 2 above. In one case, a very clear hydraulic jump is formed whereas in the other a series of waves have formed. The other photos (not printed) show similar conditions at depths of 16 feet and 8 feet above the crest for the same model. Discharges corresponding to these different depths of flow were measured on a Rehbock weir installed in the flume and the depth of flow at each stage

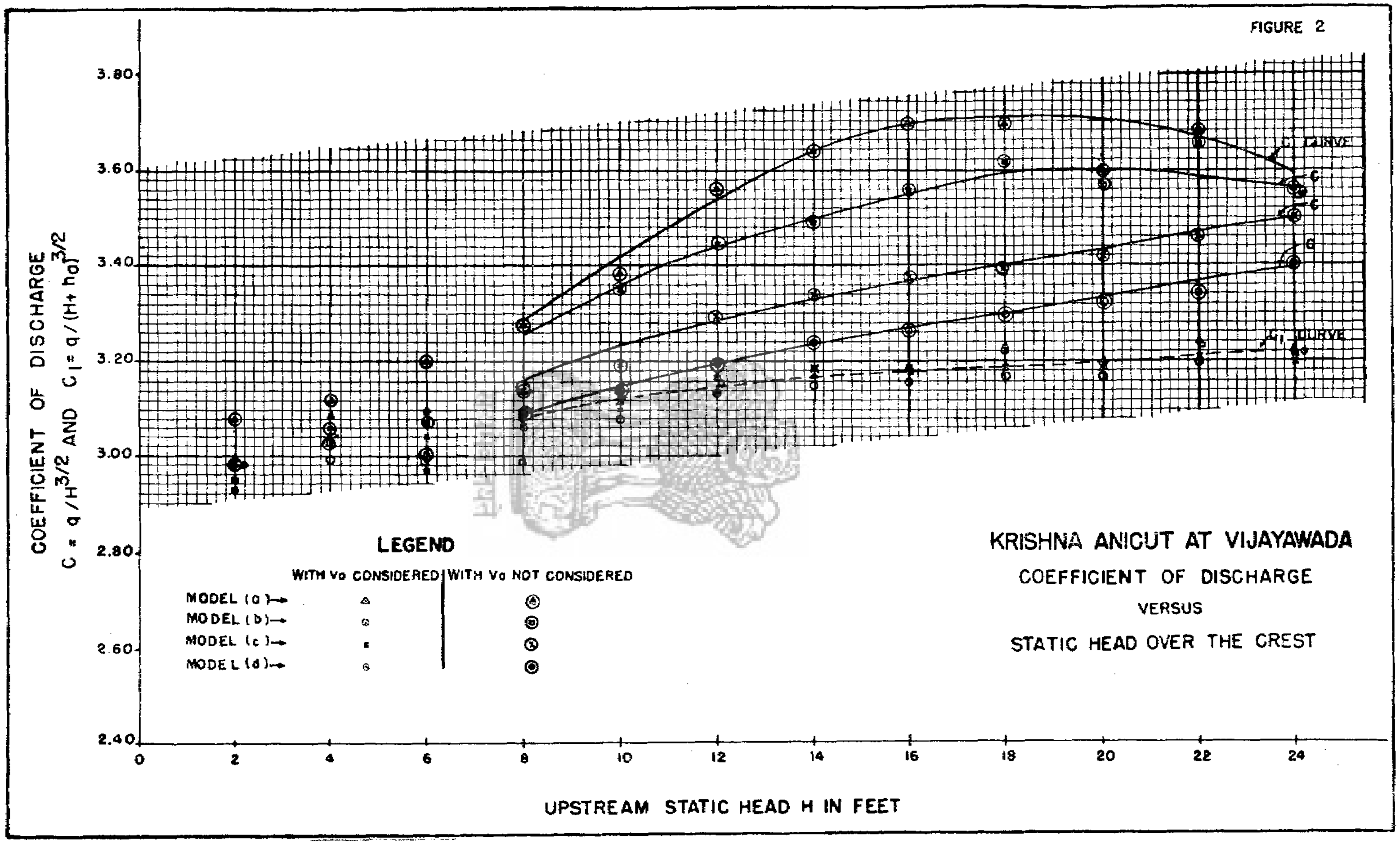
FIGURE 1



KRISHNA ANICUT AT VIJAYAWADA

U/S WATER LEVEL Vs. D/S WATER LEVEL
FOR MODULAR LIMIT

FIGURE 2



KRISHNA ANICUT AT VIJAYAWADA
COEFFICIENT OF DISCHARGE
VERSUS
STATIC HEAD OVER THE CREST

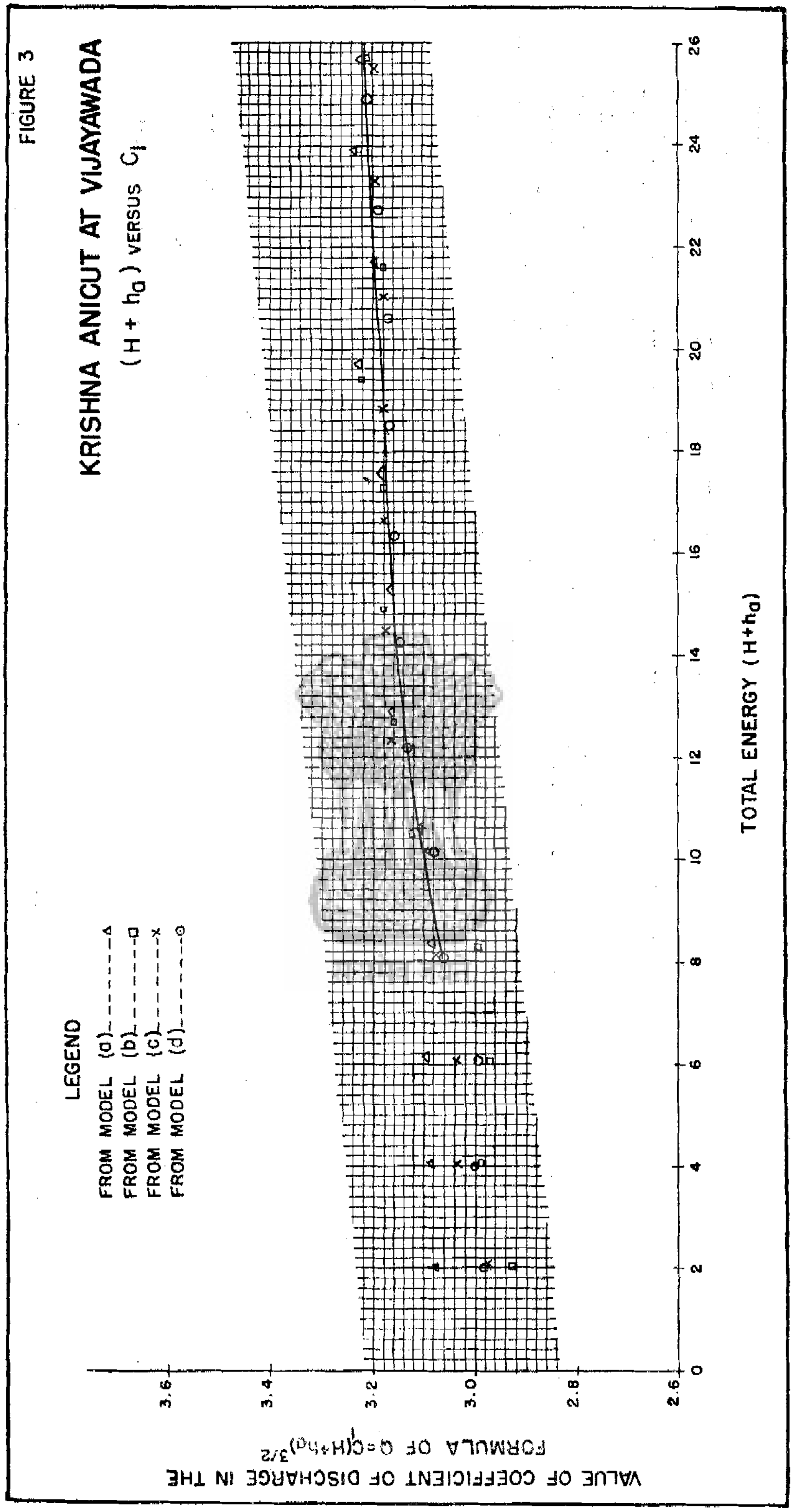


FIGURE 3

KRISHNA ANICUT AT VIJAYAWADA

$(H + h_a)$ VERSUS C_1

was measured on the upstream gauge cross-section in the case of the mobile bed models. The velocity of approach in the flume was calculated from the observed discharge intensity and the depth of flow from the equation $h_a = \alpha v^2/2g$. In this equation α is a coefficient to allow for non-uniform distribution of velocities in the approach channel. Its value can be determined experimentally. It increases as the ratio :

$$\frac{\text{Static head over crest } H}{\text{Height of crest above upstream channel bed } P}$$

decreases. Its average value has been found to be 1.09 and this was used in the present calculations shown in Tables 1 to 4. This value actually changes from 1.028 to 1.1 according to the variation of the ratio of head over the crest to height of weir *i.e.* H/P. This variation introduces a maximum error of 0.6% in the computation of value of C_1. The total energy head upstream was then computed. The coefficients of discharge C *and* C_1 in the equations $q = CH^{3/2}$ *and* $q_1 = C_1 (H+h_a)^{3/2}$ were then computed. The accompanying Tables 1 to 4 show the various values obtained in the experiments on the four models detailed in paragraph 3 above.

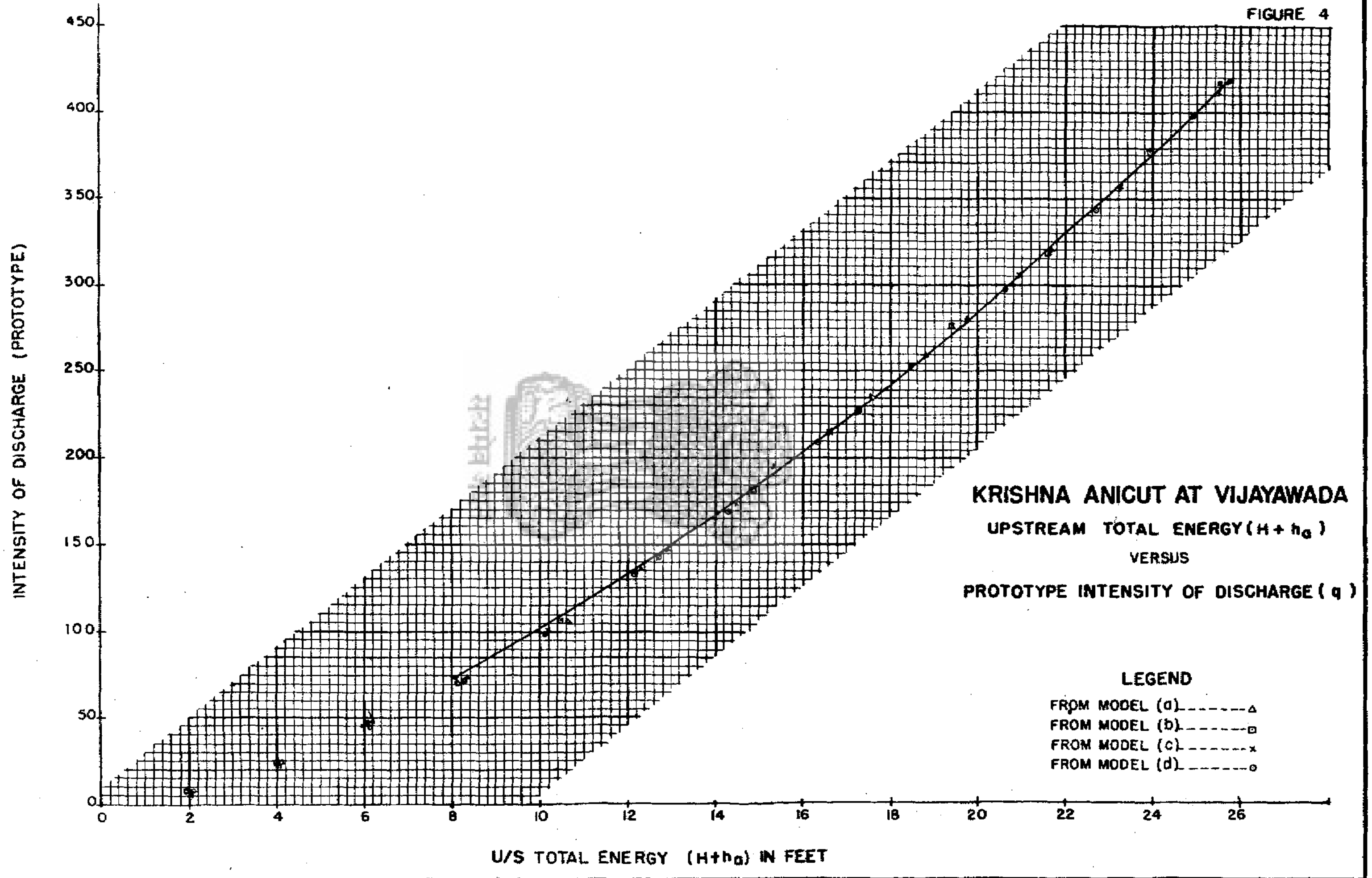
4. *Discussion of results* :

4.1. Figure 1 shows the graphs connecting the upstream and the downstream water levels at the limit of modularity as observed in the four models. In the case of mobile bed models (*a*) and (*b*), which were identical except for a change in scale the modular limit curve is seen to practically coincide, indicating that as far as the modular limit is concerned, no scale effect is perceptible.

It will be seen in Figure 1 that the modular limit curve for the models (*a*) and (*b*) *i.e.*, where the upstream bed was at R.L. 39.55, is the top most curve which gives a comparatively higher modular limit throughout the range. Next is the curve from the model (*c*) *i.e.*, when the upstream bed was at R.L. 27.22 and the lowest curve is from model (*d*) *i.e.*, when the upstream bed was at R.L. 17.22. As will be indicated later, the lowering of the upstream bed had an effect of reducing the total energy upstream and hence the discharge intensity corresponding to a fixed value of the upstream water level and naturally this resulted in lowering the downstream water level at the limit of modularity.

4.2. The coefficients C_1 in the equation $q_1 = C_1 (H+h_a)^{3/2}$ as computed for the various models as shown in the Tables 1 to 4 have been plotted against static head H in Figure 2 (lowest curve). It may well be seen that one single curve could be drawn to cover all the observations though the upstream bed levels differed to a large extent. This urvebroadly indicates that the cofficient C_1 is increasing with increase in head over the crest. The coefficient actually varies according to the total energy head upstream of the weir. Such a plotting is represented in Figure 3. The plotting in Figure 2 however, has been prepared, only because it is the static head over the crest *i.e.*, the upstream water level, which is primarily observed in the prototype. Plotting

FIGURE 4



KRISHNA ANICUT AT VIJAYAWADA
UPSTREAM TOTAL ENERGY ($H + h_a$)
VERSUS
PROTOTYPE INTENSITY OF DISCHARGE (q)

of such a curve as in Figure 2 can be justified only because the range of change in total energy corresponding to a particular value of the static head being very small, the coefficient could be assumed to be fairly constant through this range. The actual change that takes place in this range can very well be seen from Figure 3, which represents the proper relation.

The value of the coefficient C in the formula $q=CH^{3/2}$ also have been computed for the four models and given in Tables 1 to 4 and the same have been plotted in Figure 2. It will be seen that there is an independent curve for each value of the upstream bed level, the coefficient C having lower values for lower levels of the upstream bed. In the case of mobile bed models (*a*) and (*b*), the depth of flow upstream of the weir was not naturally constant due to mobility of the bed and hence the points do show some scatter. However, the observations indicated that the depth of flow below the crest could be taken as fairly constant up to a static head of 16 feet over the crest. For higher values of the static head, the depth was found to increase due to bed scour resulting in a little drooping down of the plotted curve.

A study of these different curves indicates that the primary cause of getting lower coefficient for lower upstream bed levels could be that, for a fixed value of the upstream static head, there is a reduction in the velocity of approach resulting from lowering of the upstream bed level. This reduction in the approach velocity naturally reduces the corresponding head h_a and hence the resultant total energy upstream of the weir, resulting in appreciable reduction of the intensity of discharge. The points on the curve showing intensity of discharge plotted against total energy as shown in Figure 4 indicates this behaviour correctly.

It will be clear that the lower the upstream bed, the lower the discharge intensity though the coefficient C_1 discussed in paragraph 4.2 is practically the same because the range of change of total energy is very small.

4.3. Result of models (*a*) and (*b*) indicate that the values of coefficient C_1 as obtained for static heads between 8 feet and 24 feet are practically the same. However, below a static head of 8 feet the coefficients obtained from 1/25 scale model are slightly lower than those obtained from 1/40 scale model, this difference being 4% and 5% corresponding to static heads of 4 feet and 2 feet respectively. The difference corresponding to head of 6 feet is somewhat less.

These differences in the values of the coefficients may be attributed to a scale effect caused by

surface tension,

viscosity, and

velocity distribution in the approach channel. According to Prof. A.H. Gibson* the coefficient worked out from 1/50 scale model is likely to be 2.8% too high as compared to the prototype.

**Scale Models in Hydraulic Engineering by J. Alen.*

According to Knidsvater and Carter*, the relative influence of viscosity and surface tension increases as head on the weir and the size of the weir decreases. They, however, have not specified either the minimum head on the weir or the limit of the size of the weir, at which these effect predominate. The curves plotted in the same paper, however, indicate that the minimum static head should be between 0.2 and 0.3 of a foot.

"Engineering hydraulics" edited by Hunter Rouse states that the spillway models should be scaled so that normal heads over the crest exceed 3 inches *i.e.*, 0.25 of a foot.

The above references indicate that model observations to be unaffected by scale effect, the minimum head over the crest should be about 0.2 foot.

In the case of 1/40 scale model of depths below 8 feet head over the crest will thus be subject to variation in coefficients. In the case of 1/25 scale model, depths of 5 feet and below are likely to indicate change in coefficients, this change being of the order of 5% to 4%.

4.4 For carrying out the above experiments the upstream water levels have been observed on the models and the corresponding downstream water levels at the modular limit also have been observed, but the method of computing the coefficient C_1 in the paragraph 4.2 above does not depend on the downstream water level so far as it is lower than the modular limit.

In the case of the equation $Q=[(h+h_a)^{3/2}-h_a^{3/2}]+C_2.L.d\sqrt{2g(h+h_a)}$, the symbol h has altogether a different meaning, it being equal to the difference between upstream and downstream water levels. No data about these corresponding levels is available from the prototype. However, within the modular limit, *i.e.*, when the downstream water levels are lower than those at the modular limit, the values of the coefficients will change along with the downstream water levels if the discharge intensity is assumed to be constant. Alternatively, if the values of C_2 as given in the Annexure II are taken there will be a considerable change in the discharge intensities calculated by the above formula, *vide* Appendix-1. hence the verification of the accuracy of the formula is not possible unless the actual downstream and corresponding upstream water levels from the prototype are known.

5. *Calculation of discharge* :

Various curves giving values of the discharge coefficients have been discussed in paragraph 4. It has been seen that the coefficient of discharge actually varies according to the total energy available upstream of the weir. From this point of view the ideal curve to plot and use for computing the discharge would be that shown in Figure 3. But for its actual use for computing the discharge flowing over the weir, it would be necessary to estimate the value of the head corresponding to the velocity of approach.

**Discharge characteristics of rectangular thin-plate weirs by Knidsvater and R.W. Carter, A.S.C.E. paper No.* 1453.

This can be estimated by actual measurements of velocities in the prototype. Alternatively, if instead of velocities the depth of flow upstream of the weir is measured, the actual total energy upstream of the weir can be computed by successive approximations.

The lowest curve in Figure 2 which indicates the relation between coefficient C_1 (taking into account head due to approach velocity) and the static head H could also be used by making successive approximations if the depth of flow is measured in the prototype. Both, this curve and the one discussed in the previous sub-para, have that advantage of being fully applicable for all depths of flow upstream.

The other alternative would be to use a curve similar to the upper four curves in Figure 2 *i.e.*, a plot of coefficient C against static head upstream of the weir. These curves have a disadvantage that a separate curve is required for each level of the upstream bed *i.e.*, its depth below the weir crest. As, such curves would not be available for all different depths, it will be a matter of approximation only for all intermediate points. Anyway, even for making such an approximation, it would be necessary to evaluate the bed level upstream by measurement of depth of flow on the prototype. Once such depth is measured, it would be better to use a more exact curve as in Figure 3 instead of trying for approximation as in the case of upper four curves in Figure 2.

6. *Conclusions*:

Studies so far carried out on the four models of the Krishna anicut have indicated the following :

(1) (*i*) The weir is having fairly high modular limit and the curves connecting the upstream water level against the corresponding downstream water level at the modular limit are shown in Figure 1.

(*ii*) The modular limit is lowered with the lowering of the upstream bed due to reduction in the corresponding intensity of discharge.

(*iii*) No scale effect was observed in so far as the limit of modularity is concerned.

(2) (*a*) *For equation* $q_1 = C_1(H+h_a)^{3/2}$

(*i*) The coefficient of discharge C_1 was found to vary according to the total energy available upstream of the weir, *vide* Figures 2 and 3.

(*ii*) With the coefficient C_1 computed from the total energy upstream of the weir a single curve could include all the points irrespective of the upstream bed level.

(*b*) *For equation* $q = C\ H^{3/2}$

(*i*) The coefficient C also was found to increase with the increase of head.

(*ii*) Corresponding to each value of the upstream bed a separate curve was required to be plotted connecting C with H, *vide* Figure 2.

(3) (*i*) Results obtained from 1/40 scale model would be satisfactory for all heads over crest more than 8 feet. For lower heads an error of about 5% would be introduced, the error being higher for lower heads.

(*ii*) Results of 125 scale model would be satisfactory for heads of 5 feet and more For lower depths the coefficients in the prototype are likely to be somewhat lower.

(*iii*) Lowering of the upstream bed had an effect of reducing the velocity of approach and the corresponding head h_a and this resulted in reducing the corresponding intensity of discharge and hence the coefficient of discharge C. The reduction in the coefficient C_1 was very small, as the changes in the total energy due to change in the head due to velocity of approach h_a was very small and the value of C_1 could be taken practically as constant over this small range of change of the total energy.

(*iv*) For correct estimation of discharge intensity, the upstream bed level will have to be ascertained in the prototype for each observation : With this depth ascertained in the prototype, a suitable coefficient may be selected either from Figure 3 or the lowest curve in Figure 2 as discussed in paragraph 5.

4. The formula $Q = 3.1\,L\,[(h+h_a)^{3/2} - h_a^{3/2}] + C_2 L d\sqrt{2g(h+h_a)}$ indicated that even below the modular limit the discharge will vary according to the difference between the upstream and downstream water levels as indicated in Appendix I.

APPENDIX

Calculations of discharge intensities by the equation

$$Q=3.1L[(h+h_a)^3 - h_a\ ]+C_2Ld\sqrt{2g(h+h_a)}.$$

The calculations presented here are only for an upstream water level of 57.22 assuming different water levels on the downstream side. All these, therefore, correspond to static head H of 10 feet and the intensity of discharge as found in the experiment of model (a) was 107 cusec per foot run. Different values of the intensity of discharge q as obtained by calculating from the present equation are given in the following table.

Serial No.	*Down stream water level*	*h*	*d*	*C2 as given in the Annexure V*	*q when h_a= 0·6 feet as per model(a)*	*q when h^a=0·3 as given in the Annexure V*
1	2	3	4	5	6	7
1	56·94	0·28	9·72	0·90	66·90	54·25
2	55·94	1·28	8·72	0·84	87·12	79·65
3	54·94	2·28	7·72	0·75	92·45	87·11
4	53·94	3·28	6·72	0·66	92·33	87·97

The values of q in the columns 6 and 7 of the above table firstly indicate the difference in the discharge intensity q likely to be caused due to change in the head corresponding to the velocity of approach, both as given in the Annexure II and as found on the model (a). Another point to note is the great difference that all these calculated intensities are showing, from the discharge intensity actually observed on the model (a), namely 107 cusecs per foot run. The Tables 2 to 4 indicate that due to lowering of the upstream bed, the discharge intensity also was lowered from 107 to 98.37 cusecs, the actual discharge intensities and the depths of the upstream channel bed below the weir crest being as follows :

P *i.e., the depth of upstream channel bed below the weir crest* (feet)	*Discharge intensity cusecs per foot run*	*Remarks*
8	107	Model (a)
9·50	106	Model (b)
20	100·89	Model (c)
30	98·37	Model (d)
		all corresponding to upstream static head of 10 feet over the weir crest.

It will thus be seen that the calculated discharge intensities are varying to a large extent according to the variation of the downstream water levels.

TABLE 1

KRISHNA ANICUT AT VIJAYAWADA.

Model (a) :—1/40 scale geometrically similar model with upstream bed at +39·55 (mobile bed)

Serial No.	*Upstream static head H*	*Intensity of discharge q*	*Up-stream water level*	*Down-stream water level at limit of modular range*	*Upstream depth of scour below crest.*	*Velocity of approach Va*	*Head due to velocity of approach* $h_a = \propto \times V^2a/2g$ $(\propto = 1.09)$	*Total energy upstream* $(H+ha)$	H_1 *Down-stream static head*	$C_1 = \frac{q}{(H+ha)^{3/2}}$	$C = \frac{q}{H^{3/2}}$	*Drowning ratio* $\frac{H_1}{H+ha} \times 100$ *(percentage)*
1	2	3	4	5	6	7	8	9	10	11	12	13
1	2.00	8.70	49.22	48.82	7.20	0.95	0.01	2.01	1.60	3.08	3.08	79.90
2	4.00	25.00	51.22	50.90	8.40	2.01	0.07	4.07	3.68	3.09	3.12	90.40
3	6.00	47.00	53.22	52.90	8.55	3.23	0.18	6.18	5.68	3.10	3.20	92.00
4	8.00	74.00	55.22	54.98	8.00	4.66	0.36	8.36	7.76	3.09	3.27	92.80
5	10.00	107.00	57.22	56.94	8.00	5.96	0.60	10.60	9.72	3.11	3.38	91.70
6	12.00	148.00	59.22	58.98	8.00	7.38	0.93	12.93	11.76	3.168	3.56	90.90
7	14.00	190.00	61.22	60.98	7.00	9.05	1.39	15.39	13.76	3.170	3.63	89.50
8	16.00	236.00	63.22	63.06	8.80	9.65	1.53	17.58	15.74	3.185	3.69	89.50
9	18.00	282.00	65.22	64.90	10.00	10.05	1.71	19.71	17.68	3.230	3.69	89.50
10	20.00	321.00	67.22	66.82	12.70	9.80	1.63	21.63	19.60	3.20	3.59	90.50
11	22.00	380.00	69.22	68.90	14.00	10.55	1.38	23.88	21.68	3.24	3.68	90.70
12	24.00	419.00	71.22	70.82	18.40	9.99	1.68	25.68	23.60	3.22	3.56	91.80

TABLE 2

KRISHNA ANICUT AT VIJAYAWADA

Model (b) :—1/25 scale geometrically similar model with upstream bed at +39·55 (mobile bed)

Serial Number	*Upstream static head* H	*Intensity of discharge* q	*Up-stream water level*	*Down-stream water level at limit of modular range*	*Depth of scour below crest*	*Velocity of approach* V_a	*Head due to* V_a $h_a=\alpha V_a^2/2g$ $(\alpha=1.09)$	*Total energy upstream* $(H+ha)$	*Down-stream static head* H_1	$C=\dfrac{q}{(H+ha)^{3/2}}$	$C=\dfrac{q}{H^{3/2}}$	*Drowning ratio* $\dfrac{H_1\times 100}{H+h_a}$
1	2	3	4	5	6	7	8	9	10	11	12	13
1	2·00	8·34	49·22	48·82	9·28	0·73	0·009	2·009	1·60	2·93	2·95	80·00
2	4·00	24·25	51·22	50·82	9·00	1·97	0·059	4·059	3·60	2·97	3·03	88·75
3	6·00	45·20	53·22	52·82	9·00	3·01	0·153	6·153	5·60	2·97	3·07	91·20
4	8·00	71·50	55·22	54·94	9·00	4·21	0·299	8·299	7·72	2·99	3·14	93·00
5	10·00	106·00	57·22	56·94	9·50	5·43	0·498	10·498	9·72	3·12	3·35	92·75
6	12·00	143·00	59·22	59·01	10·00	6·50	0·714	12·714	11·79	3·16	3·44	92·75
7	14·00	183·00	61·22	60·84	11·00	7·32	0·906	14·906	13·62	3·18	3·49	91·50
8	16·00	228·00	63·22	62·82	10·50	8·60	1·257	17·257	15·60	3·18	3·56	90·50
9	18·00	276·00	65·22	64·82	12·00	9·20	1·430	19·430	17·60	3·22	3·61	90·50
10	20·00	319·00	67·22	66·84	12·50	9·80	1·626	12·626	19·62	3·18	3·57	90·60
11	22·00	377·00	69·22	68·82	13·00	10·77	1·960	23·960	21·60	3·23	3·65	90·20
12	24·00	418·00	71·92	70·94	18·00	9·96	1·680	25·680	23·72	3·21	3·55	92·40

TABLE 3

KRISHNA ANICUT AT VIJAYAWADA

Model (c) :—1/40 scale geometrically similar model with upstream bed laid 20 feet below the crest *i.e.*, at + 27.22 and made rigid.

Serial number	*Upstream static head* H	*Intensity of discharge* q	*Upstream water level*	*Down stream water level at limit of modular range*	*Velocity of approach* V_a	*Head due to* V^a $h_a = \frac{\alpha \times V_a^2}{2g}$ $(\alpha = 1.09)$	*Total energy upstream* $H+h_a$	*Down stream static head* H_1	$C_1 = \frac{q}{(H+h_a)^{3/2}}$	$C = \frac{q}{H^{3/2}}$	*Drowning ratio* $\frac{H_1}{H+h_a} \times 100$
1	2	3	4	5	6	7	8	9	10	11	12
1	2.00	8.42	49.22	48.86	0.37	0.0023	2.002	1.64	2.98	2.98	81.92
2	4.00	23.73	51.22	50.66	0.99	0.0165	4.0165	3.44	2.99	2.99	85.65
3	6.00	45.24	53.22	52.66	1.74	0.051	6.051	5.44	3.04	3.08	89.90
4	8.00	71.12	55.22	54.66	2.54	0.110	8.11	7.24	3.08	3.14	89.27
5	10.00	100.89	57.22	56.26	3.36	0.190	10.19	9.04	3.10	3.19	88.71
6	12.00	136.95	59.22	58.26	4.28	0.310	12.31	11.04	3.17	3.29	89.68
7	14.00	174.52	61.22	60.26	5.13	0.450	14.45	13.04	3.18	3.33	90.24
8	16.00	215.45	63.22	62.26	5.98	0.606	16.606	15.04	3.18	3.37	90.57
9	18.00	259.25	65.22	64.26	6.82	0.788	18.788	17.04	3.18	3.39	90.70
10	20.00	306.00	67.22	66.16	7.65	0.993	20.993	18.94	3.18	3.42	90.22
11	22.00	357.67	69.22	68.06	8.52	1.230	23.23	20.84	3.20	3.46	89.71
12	24.00	412.29	71.22	69.92	9.37	1.490	25.49	22.70	3.20	3.50	89.05

TABLE 4

KRISHNA ANICUT AT VIJAYAWADA

Model (d) :—1/40 scale geometrically similar model with upstream bed laid 30 feet below crest *i.e.*, at +17·22 and made rigid

Serial number	Upstream static head H	Intensity of discharge q	Upstream water level	Down stream water level at limit of modular range	Velocity of approach V_a	Head due to V_a $h_a = \frac{\alpha V_a^2}{2g}$ $(\alpha = 1.09)$	Total energy upstream $(H+h_a)$	Down stream static head H_1	$C_1 = \frac{q}{(H+h_a)^{3/2}}$	$C = \frac{q}{H^{3/2}}$	Drowning ratio $\frac{H_1}{H+h_a} \times 100$
1	2	3	4	5	6	7	8	9	10	11	12
1	2.00	8.42	49·22	48.86	0.26	0.001	2.001	1.64	2.98	2.98	81.96
2	4.00	23·72	51·22	50.66	0.70	0.008	4.008	3·44	2·99	2·99	85·83
3	6.00	44·30	53·22	52.66	1·23	0.03	6.03	5·44	2·99	3·00	90·22
4	8.00	70.06	55·22	54·46	1.84	0.06	8.06	7·24	3·06	3·09	89·83
5	10.00	98·37	57·22	56.26	2·46	0.10	10.10	9.02	3·08	3·13	89·50
6	12.00	132·75	59·22	58.06	3.16	0.17	12.17	10.84	3·13	3·19	89·07
7	14·00	169.28	61.22	60.06	3·85	0.25	14·25	12.84	3·15	3·23	90.10
8	16.00	208·78	63·22	62.06	4·54	0·35	16·35	14·84	3·16	3·26	90·76
9	18.00	251·23	65·22	63.86	5·23	0·46	18.46	16.64	3·17	3·29	90.14
10	20.00	296.62	67·22	65.66	5·93	0.60	20.60	18·44	3·17	3·32	89·51
11	22.00	344·69	69·22	67.66	6.63	0·74	22·74	20·44	3·19	3·34	89·89
12	24.00	389.65	71·22	69.66	7·38	0·92	24·92	22·44	3·21	3·40	90·05

PART III

KRISHNA ANICUT AT VIJAYAWADA

EXPERIMENTS BY THE INDIAN INSTITUTE OF SCIENCE, BANGALORE

1. *The Problem :* The Krishna-Godavari Commission posed for investigation the following problems regarding the flow of the Krishna river over the anicut at Vijayawada :—

(*i*) Determine, by model experiments, the modular range for the section of the Krishna Anicut at Vijayawada (as it existed before the construction of the Krishna Barrage) for all values of head on crest up to 24 feet, in other words, to ascertain for each value of *H* (4 feet, 6 feet ....up to 24 feet), the highest downstream water level at which the flow over the anicut is not affected, in any way, by the downstream water level.

(*ii*) Within the modular range and for different values of *H*, determine—

(*a*) the values of the coefficient *C* in the equation,

$$Q=C\ L\ H^{3/2} \qquad (1)$$

(*b*) the values of the coefficient C_1 in the formula

$$Q=C_1\ L\ (H+h_a)^{3/2} \qquad (2)$$

(*iii*) In the non-modular range, for different drowning ratios, determine the value of the coefficient in the formula,

$$Q=C\ L\ H^{3/2} \qquad (3)$$

(*iv*) Verify on the basis of model experiments, the accuracy of the following formula for the range of conditions when the downstream water level is higher than the crest level of the anicut :

$$Q=3.1\ L\ \{\ (H+h_a)^{3/2} - h_a^{3/2}\ \} + C_2 L \propto \sqrt{2g\ (h+h_a)} \qquad (4)$$

(*v*) In determining the coefficients of discharge *C* and C_1, it may be stated whether there is any "scale effect" ;

(*vi*) Determine whether there is any effect of the level of the upstream bed on the value of the coefficient of discharge ;

(*vii*) Determine with the help of a three dimensional model, the discharge over the anicut for different upstream and when applicable downstream gauge readings, and how far the prototype gauges correctly represent the depth of flow on different parts of the anicut crest.

In these equations

Q=discharge in cusecs

C, C_1 and C_2 =coefficients of discharge in the equations 1, 2, 3 and 4

L=Length of anicut in feet, at right angles to the direction of flow

H=Head over the anicut

h_a=Head due to velocity of approach

P=Depth of upstream bed below the crest of the anicut

δ =Modularity coefficient or reduction factor

d=Difference, in feet, between the water level downstream of a weir and the crest of the weir, and

g=acceleration due to gravity.

2. Experiments were accordingly undertaken and the following note gives the results obtained.

3. Theory and applicability of existing formulae to determine flow over an anicut :

(a) Classification of flow:

Flow over a weir or anicut can be broadly divided into two ranges, namely, the modular range and the non-modular range.

In the modular range, the discharge is controlled uniquely by the upstream head H. In the non-modular range, the discharge is controlled both by upstream and downstream levels. The flow in the non-modular range is characterised by the travel of disturbances from downstream of the weir to upstream of the weir, usually in the form of waves of varying lengths, amplitudes and phase sequences. The stage of the tail water at which the downstream head d comes into discharge control is known as the modularity limit. In laboratory studies, the usually uniform steady flow in the modular region gets *just* transformed into unsteady oscillatory flow, at the transition from the modular to non-modular region. This can be observed precisely. In addition, it is also possible to notice an increase in the gauge reading of the depth of flow over the weir on the upstream side without any increase in the discharge passing over the weir. If d_c is the critical tail water depth at this modular limit and H is the corresponding upstream head, then the ratio d_c/H is known as the critical modular ratio.

On the basis of the experiments carried out, Tables 5(a) and 6(a) have been prepared to give values of the coefficients C_1 and C for different values of H commencing from four feet and at intervals of every two feet as desired, in problem (i).

Problem (iv) Accuracy of the equation (4):

The discharge as calculated by using equation (4) and actually measured values of discharges are shown in Table 7. The percentage of variation in discharge between calculated and measured values are shown in :

(*b*) *Formulae for calculation of discharge in the modular range:*

In the modular range, the discharge was calculated, formerly, by the Weisbach formula

$$Q=C_3L\ [(H+h_a)^{3/2}-h_a^{3/2}] \qquad (5)$$

This formula is sometimes simplified as

$$Q=C_1L(H+h^{3/2}) \qquad (2)$$

This formula is not normally recommended since it involves a process of successive approximations in the calculation of the discharge Q for a known head H. In the simplified formula, the effect of velocity of approach is thrown into the coefficient C and equation (2) is rewritten as

$$Q=CLH^{3/2} \qquad (1)$$

(*c*) *Factors affecting value of discharge co-efficient:*

Experiments have shown that (i) in a sharp crested weir, C is a function of mainly H/P, where P is the height of the crest of the weir above the floor of the approach channel, (ii) in a broad crested rectangular weir with vertical drops both at upstream and downstream ends, C is a function mainly of H/P and H/B where B is the width of the weir (*i.e.*, length along the direction of flow), (iii) in an embankment section, that is with slopes both on the upstream and downstream sides, C is a function mainly of H/P, H/B, S_u and S_d where S_u and S_d are the upstream and downstream slopes respectively.

(*d*) *Formula for calculation of discharge in non-modular range:*

The discharge in this case is given by the formula

$$Q=C\gamma LH^{3/2} \qquad (3)$$

where γ is a reduction factor.

In sharp crested weirs γ is known to be a unique function of d/H. The critical modular ratio d_c/H takes a constant value for a given sharp crested weir, irrespective of the head H.

In broad crested weirs, with or without upstream and downstream slopes, γ is a function of H/B, S_u and S_d and the modular ration d_c/H is a function of H/B, S_u and S_d.

(*e*) *Other formulae:*

A complete list of formulae have been published in King's 'Handbook of Hydraulics' U. S. Geological Survey Paper No. 150 on "Weir Experiments, coefficients and formulas" and in many other papers in various Journals published during the past ten years.

The Indian Institute of Science has contributed two papers on this subject. Many theses, *e.g.*, (i) "Submerged broad crested weirs" by James, B. Conveyer (1950), (ii) "Study of discharge characteristics of submerged weirs" by James R. Marble, (iii) "Studies of flow characteristics of submerged dams" by J. N. Bradley (1943), (iv) "Submerged weir as a measuring device" by Cox Glen (1928), (v) "Discharge characteristics of submerged spillways", M. S. thesis, Colorado (1951) have been published, including those of Kindsvater (1959) and Neogi (1960).

All the formulae mentioned or derived by various workers are the result of observations conducted under idealised conditions of flow in laboratory flumes. Some have been the results of measurements made on a few weirs built on the rivers. If any of these formulae has to be applied to the Krishna anicut, *it will be accurate only to the extent* the model of the laboratory set-up or of the particular prototype is geometrically similar to the Krishna Anicut. In view of these limitations, it was considered desirable to undertake fresh experiments in the laboratory on two-dimensional and three-dimensional models.

4. *Experimental set up of 2-dimensional models*:

A geometrically similar model of the Krishna Anicut was put up in a two feet wide rectangular flume to satisfy the requirements of Froudian similarity first to the scale of 1 in 40, and then to the scale of 1 in 50. To ascertain the effect of 'scale' on the values of coefficient of discharge, further experiments were also made in a model to the scale of 1 in 30. The discharge passing over the model was measured by a rectangular sharp crested weir, with suppressed end contractions, for which the discharge was given by the following modified Rehbock formula. This formula was worked out by the Grenoble laboratories in France who supplied the weir for accurate work in our laboratory,

$$Q=\tfrac{2}{3}\, C_d \sqrt{2g}\, LH^{3/2} \qquad (6)$$

where

$$_d = 0.605 + 0.08\ H/P$$

The upstream and downstream heads were measured by point gauges (having a least count of 1/144 inch) located at a distance of three feet from the crest of the anicut on the upstream side and 2·5 feet from the toe of the anicut on the downstream side. Proper stilling well arrangements for observations of gauges were made. The zero of the anicut crest was determined by running a flow over it and gradually reducing the flow till it finally stopped. The reading of the water at this stage was taken. To ensure uniformity of flow, proper grid screens had been installed far away from the model. The model was constructed with masonry, finished with smooth cement mortar, correct to the given section. All other necessary precautions were taken to get accurate values.

5. *Problem* (*i*) *Determination of modular limit and Problem* (*iii*) *Determination of the coefficient in equation* (3) *in the non-modular range-2 dimensional model* :

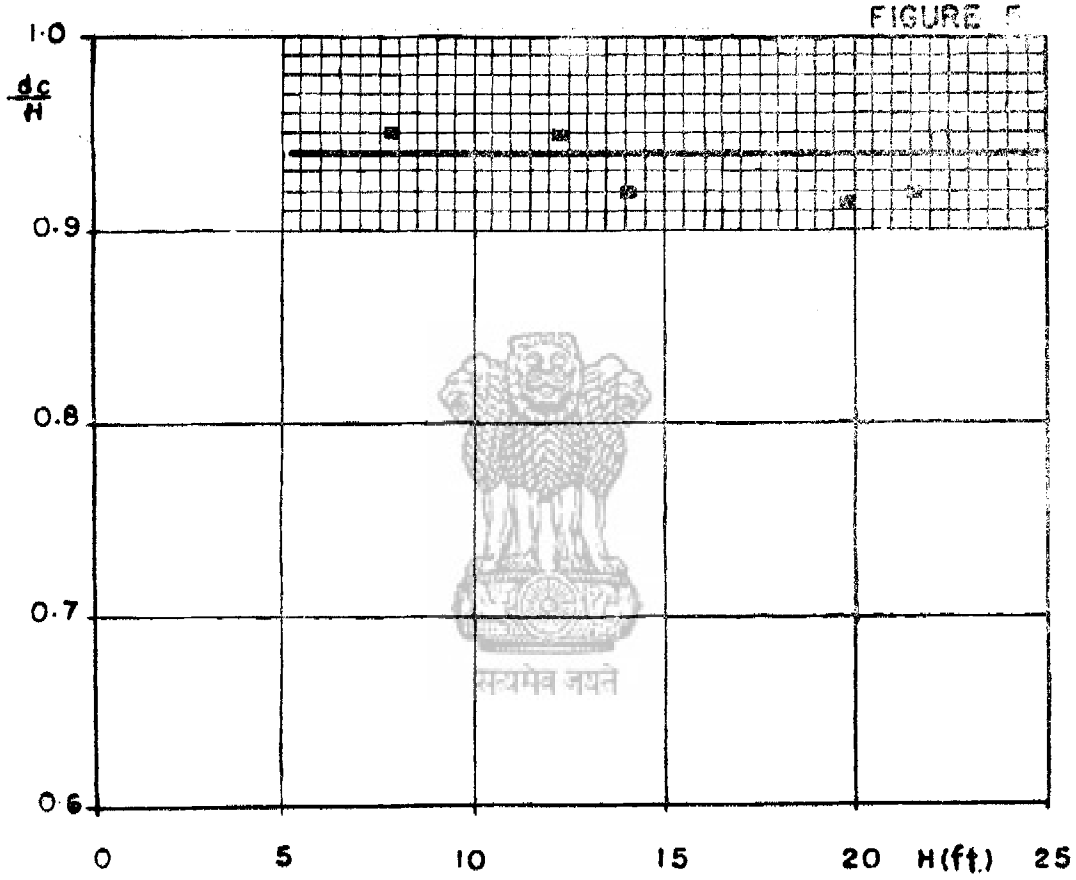
For these studies, a particular discharge was let into the flume. The downstream depth was raised by small increments from the modular range into the modular limit. As already mentioned, the critical depth d_c for non-modular flow was detected when simultaneously rise in upstream gauge and oscillations in upstream flow were noticed. The results of seven sets of experiments have not been recorded here since they are not believed to be sufficiently accurate. For each set of experiments, the upstream depth of flow was kept constant. The tail water depth in the flume was raised by increasing the height of the baffle (called tail-gate hereafter) at the end of the flume. In a river, for a particular depth of flow upstream of the anicut, there is only one corresponding depth of flow (called tail water depth in this note) downstream of the anicut, which is governed by the cross-section, depth, and slope of the river downstream and also of any obstructions to flow like bridges etc., But in a model the downstream depth can be varied by placing a small obstruction in the form of a rectangular gate across the flume. By raising the height of this gate, the depth of water is also increased. By manipulating the heights of this tail gate that is for different values of d, the values of γ for several ratios of d/H for a constant value of H can be and were obtained in a set of experiments. In a second set, for another constant value H of the upstream flow over the anicut, the values of γ for practically the same ratios of d/H as in the first set of experiments were obtained for altering the values of d by suitable manipulation of the tail gate. It was found from those results that for this shape of the anicut at Vijayawada γ has no fixed value in the model experiments. At the anicut site at Vijayawada γ can have only one particular value for particular values of H at the anicut. From the data available with us of actual observations of flow at Vijayawada, *there was not even a single case* when the flow was non-modular. These experiments are therefore of only academic interest. However the results of the experiments for the modularity limit are shown in Figure 5. It can be seen that the average modularity limit is equal to about 0.94 .

Problem (ii) *Determination of discharge co-efficients C_1 and C.*

From the results of the model experiments, the variation C_1 with H (for P=10 feet) given in Figure 6 and Table 5. It will be seen that C_1 increases from 2.0 at H=4.0 eet to 3.25 at H=25 feet.

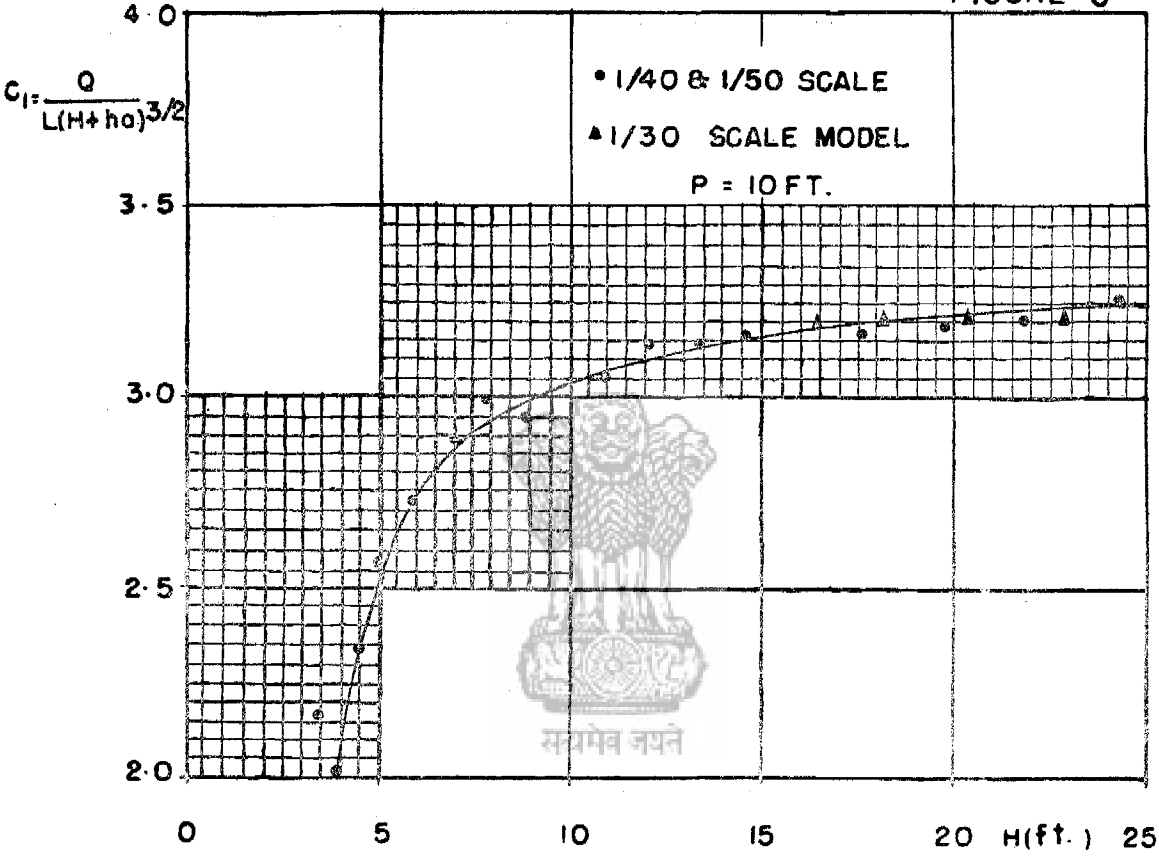
The variation of C with H (for the same case of P=10 feet) is shown in Figure 7 and Table 6. C varies from 2.0 at H=3.2 feet to 3.75 at H=25 feet.

FIGURE 5



MODULARITY STUDIES

FIGURE 6



DISCHARGE CHARACTERISTICS OF KRISHNA ANICUT

TABLE 5

Variation of C_1 with head H in 2 dimensional model with P=10 feet.

$$C_1=Q/L\,(H+h_a)^{3/2}$$

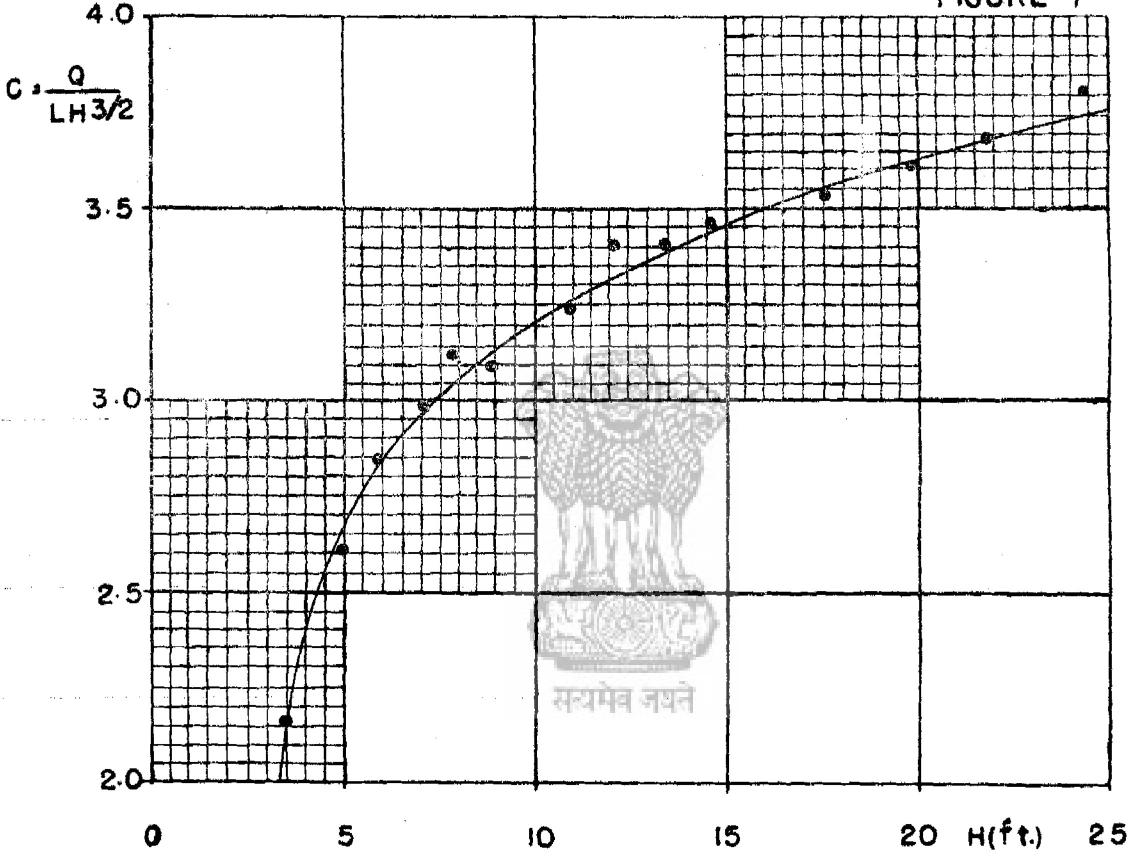
Head H	C_1	*H*	C_1
3·40	2·16	13·35	3·13
3·80	2·02	14·60	3·16
4·55	2·34	16·40	3·18
5·00	2·56	17·55	3·16
5·85	2·72	18·20	3·20
7·05	2·88	19·70	3·18
7·80	2·98	20·40	3·20
8·85	2·94	21·80	3·20
10·90	3·04	22·90	3·20
12·00	3·14	24·40	3·25

TABLE 5(a)

Working table for C_1 in the Equation $Q=C_1 L\,(H+h_a)^{3/2}$

Head *H*	C_1
4	2·07
6	2·72
8	2·97
10	3·03
12	3·09
14	3·13
16	3·17
18	3·19
20	3·21
22	3·22
24	3·23

FIGURE 7



VARIATION OF C WITH H

TABLE 6

Values of C and H for $P=10$ feet

$C=Q/LH^{3/2}$

H feet	C	H	C
3·4	2·16	12·1	3·40
5·0	2·61	13·35	3·40
5·85	2·84	14·6	3·47
7·04	2·98	17·6	3·53
7·8	3·12	19·8	3·61
8·85	3·08	21·8	3·68
10·90	3·24	24·4	3·81

TABLE 6(a)

Working table for C in the equation $Q=CLH^{3/2}$

Head H	C
4	2·40
6	2·85
8	3·06
10	3·20
12	3·32
14	3·41
16	3·50
18	3·56
20	3·62
22	3·69
24	3·73

TABLE 7

Comparison of discharge by equation (4) and I.I. Sc. model studies

Head over anicut	*Prototype*		*Modularity limit*	*Nature of flow*	*Discharge per foot by I.I. Sc. model studies q*	*Discharge by equation* (4) *q'*	*Difference* Δq $= q - q'$	$\frac{\Delta q}{q} \times 100$
	upstream *level*	down-stream *level*						
8·0	55·05	51·05	54·55	Modular	70·4	66·9	3·5	4·97
10·0	57·05	54·25	56·45	,,	100·3	84·1	16·2	16·10
12·0	59·05	57·15	58·35	,,	140·4	123·4	17·1	12·20
14·0	61·05	59·65	60·25	,,	181·5	139·4	42·1	23·20
16·0	63·05	61·65	62·15	,,	224·0	159·7	64·3	28·70
18·0	65·05	63·65	63·95	,,	271·0	185·8	85·2	31·40
20·0	67·05	65·15	65·85	,,	321·0	240·2	80·8	25·20
21·0	68·05	65·90	66·75	,,	348·0	268·2	79·8	23·00
23·0	70·05	67·25	68·65	,,	403·0	333·7	69·3	17·20
25·0	72·05	68·45	70·55	,,	456·0	408·0	48·0	10·50

It is seen that equation (4) always gives a lower discharge and the variation of the percent error with head. This is shown in Figure 8. It varies from 4.97% to 31·4%.

The use of this formula is now condemned as inaccurate. The following is an extract from "Fluid Mechanics" by Cox and Germano. "A fundamental error of analysis has persisted with many writers. These writers assume that the discharge is the sum of the discharge over a free fall weir for the portion above the downstream head elevation and the discharge from a submerged orifice for the portion between this and the elevation of the crest. *Such an assumption would require a velocity profile similar to that for a weir for the upper portion and then a constant velocity for the lower portion.* Such a requirement is quite illogical and *not true to fact*".

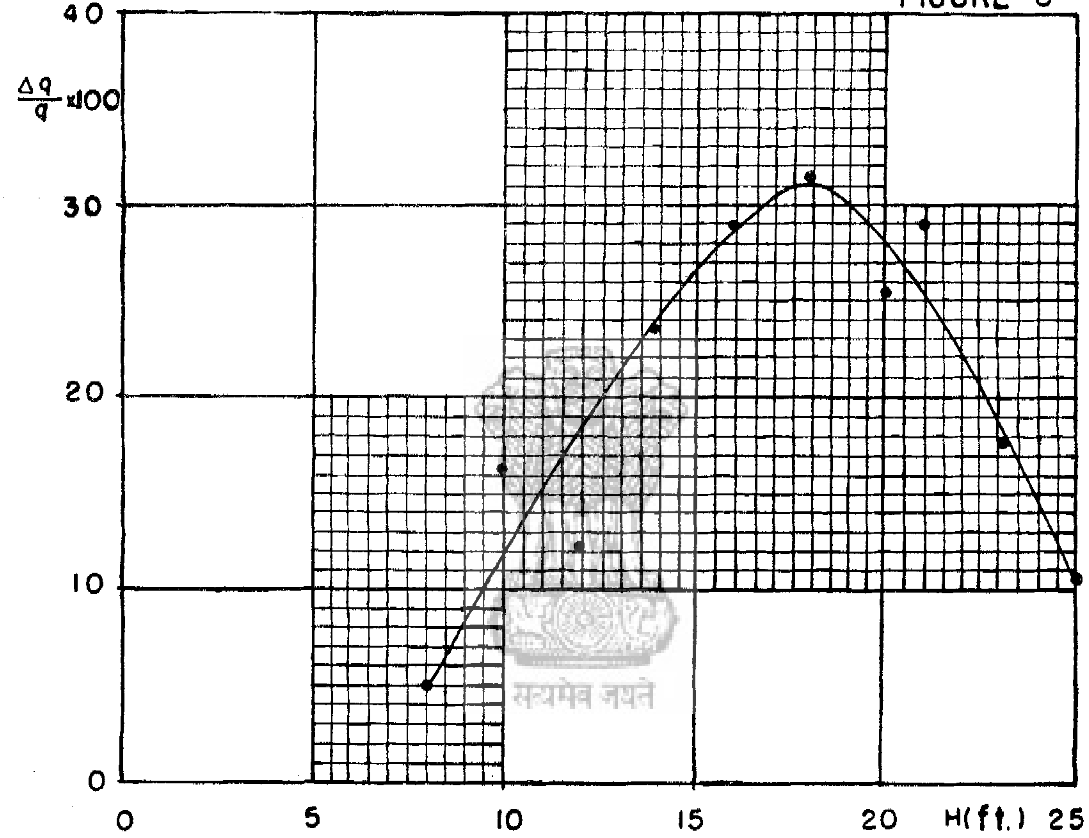
Problem (v) Scale effect in a 2-dimensional model :

To find out whether there is any appreciable scale effect, three models to 1/30, 1/40 and 1/50 scales were laid. The values of C are shown in Figure 6 from which it is seen that the scale effect in a 2-dimensional model is negligible.

Problem (vi) effect of upstream depth :

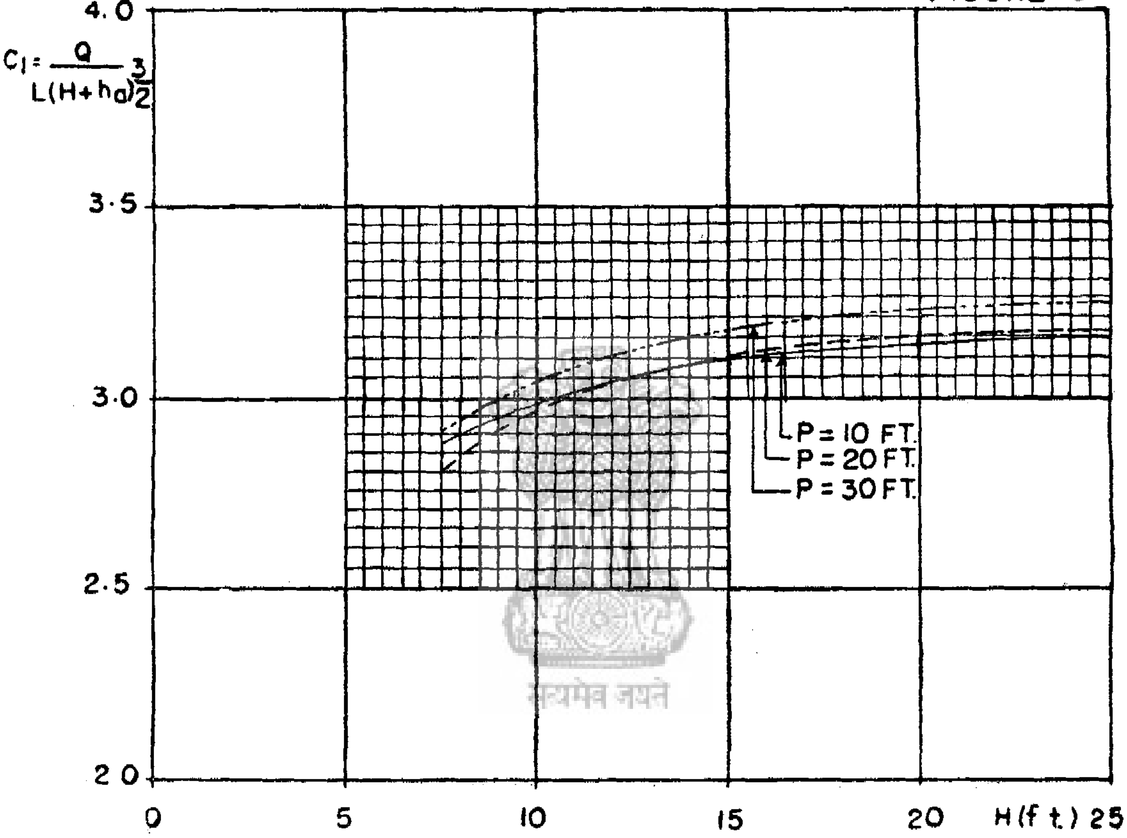
Tables 8 and 9 and Figures 9 and 10 give a comparative statement of the effect of upstream scour. Since deep scours occur at only high depths of discharge over the anicut, observations of C (coefficient of discharge) have been confined to depths of flow.

FIGURE 8



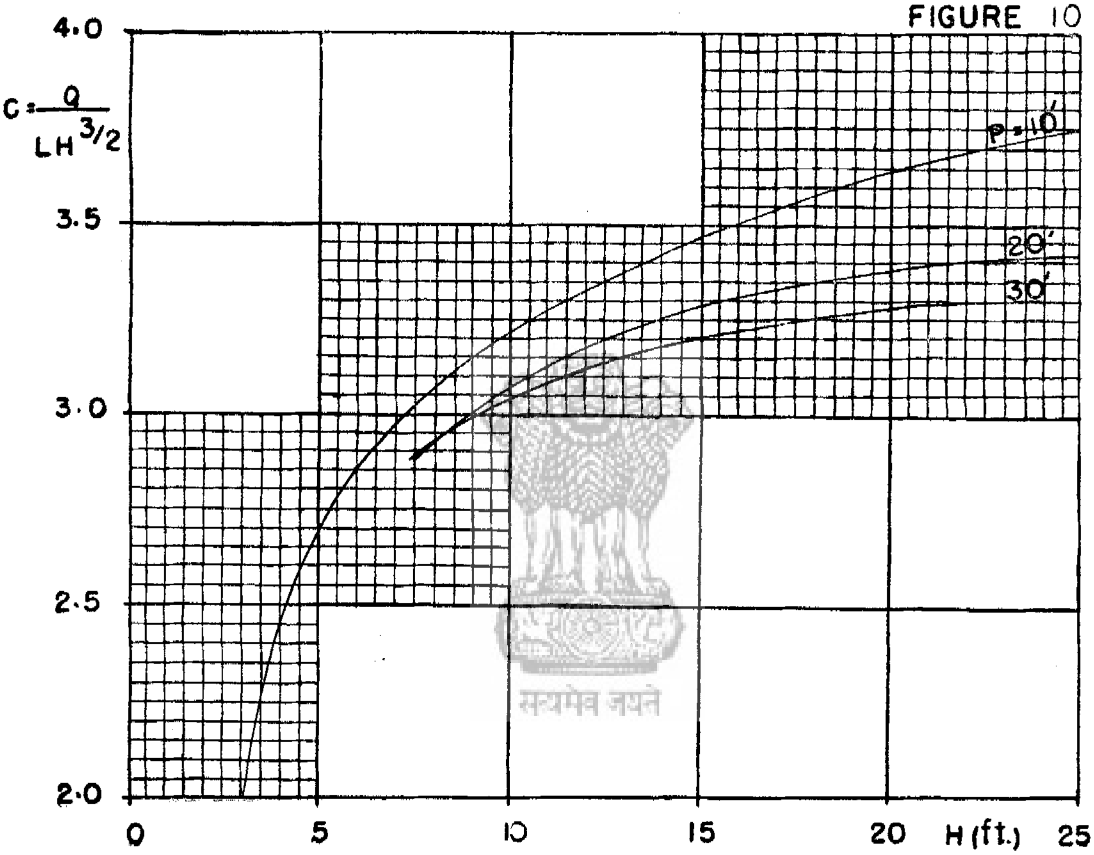
COMPARISON OF DISCHARGE COMPARED BY FORMULA(EQ.4) & MODEL STUDIES

FIGURE 9



EFFECT OF UPSTREAM DEPTH ON C_1

FIGURE 10



EFFECT OF P ON C

TABLE 8

Comparison of the variation of C_1 in Tables 5, 6 and 7: P = Depth of upstream bed below crest (scale 1/50)

Prototype head H (feet)	*Coefficient of discharge C_1*			*Percentage variation between*	
	P=**10** *feet* (*A*)	*P*=**20** *feet* (*B*)	*P*=**30** *feet* (*C*)	(*A*) *and* (*B*)	(*B*) *and* (*C*)
7·5	2·90	2·82	2·90	2·7	2·8
10·0	3·03	2·96	2·98	2·3	0·07
12·5	3·10	3·05	3·05	1·6	nil
15·0	3·16	3·10	3·09	1·9	0·03
17·5	3·20	3·13	3·11	2·2	0·07
20·0	3·22	3·15	3·13	2·2	0·07
22·5	3·23	3·16	3·14	2·2	0·07
25·0	3·24	3·16	3·15	2·5	0·03

TABLE 9

Variation of C with P

Prototype H (feet)	C			*Percentage variation between*	
	P=**10** *feet* (*A*)	*P*=**20** *feet* (*B*)	*P*=**30** *feet* (*C*)	(*A*) *and* (*B*)	(*B*) *and* (*C*)
7·5	3·00	2·87	2·88	4·3	0·33
10·0	3·20	3·07	3·03	4·0	negligible
12·5	3·35	3·20	3·13	4·5	4.1
15·0	3·46	3·28	3·19	5·2	2·7
17·5	3·55	3·33	3·25	6·2	2·4
20·0	3·63	3·38	3·28	6·9	3·0
22·5	3·70	3·40	3·30	3·1	3·0
25·0	3·75	3·42	3·30	8·8	3·5

exceeding 7.5 feet. It should not be forgotten that deep scours that may occur during high stages of flow may get filled up during its low stages.

In Figure 9 and Table 8, it is seen that the variation of C_1 (coefficient of discharge) when the scour is between 10 feet and 20 feet never exceeds 2.7%. If the depth of scour is between 20 feet and 30 feet the value C_4 varies over a very narrow range. In these calculations the effect of the velocity of the approach is considered separately by calculating h_a.

In Figures 10 and Table 9, the value of C itself (coefficient of discharge) takes within its value the effect of the velocity of approach. As the variation of the velocity of approach is function of the scour depth in relation to the total area of flow, it is seen that the value of the coefficient of discharge varies sharply when the scour is between 10 feet and 20 feet. The range of variation is from 4.0% for $H=10$ feet to 8·8% for $H=25$ feet. By adopting equation (2) in the calculations of discharge, it is possible to overcome very largely the effect of scour on the value of the coefficient of discharge.

Problem (vii) Coeffiecient of discharge in a 3-dimensional model:

Studies with 3—dimensional model:

Since the flow in the prototype was considerably oblique and the choice of the prototype gauge locations were not happy and since all the theoretical formulae available could not apply "mutatis, mutandis" to the Vijayawada anicut discharge, it was decided to put up a 3-dimensional model. Because of the restrictions of space, time, and maximum discharge available in the laboratory did not permit the construction of a geometrically similar 3-dimensional model to a fairly big size which could be free from surface tension and other effects, a model with a horizontal scale of 1 in 1000 and a vertical scale of 1 in 250 was put up inside the laboratory.

Experimental set-up:

The river corss sections were cut in plywood boards and the river bed was moulded rough with cement plaster. It was moulded rough in order to eliminate errors due to surface tension effects during runs of low heads over the anicut. The anicut was constructed in cement mortar and polished smoothly. The gauges were fixed exactly according to prototype drawings. In addition also, one upstream and one downstream gauge were installed according to standard hydraulic practice. The discharge was measured with a rectangular weir with suppressed end contractions which was initially calibrated using Rehbock's modified formula. A tail gate was provided to enable non-modular studies being made. Stilling arrangements were provided at all the required places.

The errors due to scale effect was overcome by observing similarity in the coefficients of rugosity in the model and the prototype. According to Manning's equation, the rugosity coefficient in the model should be N (m) $R^{2/3}\, Lr^{-1/2}$ times the rugosity coefficient in the prototype, where R is the hydraulic mean radius and Lr is the scale of the model. For the anicut portion a value of 0·017 for rugosity coefficient in the prototype (for rubble masonry) was assumed and the value of *n* in the model works out to 0.01 nearly. To ensure this the surface was plastered smooth. In the river portion, the value of *n* was assumed to be .035 (natural river with irregular side slopes). Its equivalent in the model works out to roughly .015 nearly and the river bed was, therefore, plastered a little rough. This will also overcome surface tension effects in the model running at low discharges.

The values of the coefficients of discharge obtained under both modular and non-modular conditions in the 3-dimensional model cannot be adopted as the flow conditions were noticed to be distorted due to vertical exaggeration in the model. Similarly these model experiments cannot give a definite answer to the last problem posed as to how far the prototype gauges correctly represent the depth of flow on different parts of the anicut crest.

PART IV

KRISHNA ANICUT AT VIJAYAWADA

EXPERIMENTS BY THE ANDHRA PRADESH ENGINEERING RESEARCH LABORATORIES, HYDERABAD

1. From some initial experiments, it was ascertained that the flow over the Krishna Anicut is modular up to 90 to 94 per cent submergence. Detailed experiments on three dimensional models were, therefore, confined to modular conditions only.

2. *Studies on the 3—dimensional model to a geometrically similar scale of* 1/120 :

This is a rigid bed model in which the river has been reproduced two miles upstream and one mile downstream of the anicut according to the river survey data of 1956. The river bed immediately upstream of the model is 10 feet below the crest of the anicut.

The heads over the anicut model were measured in stilling chambers with point gauges reading up to .001 feet and the discharges were measured over a standard Rehbock weir.

The following series of experiments were conducted on this model:

Series 1: Determination of the discharges over the anicut for various heads:

Various discharges were run and the corresponding upstream heads over the anicut were noted under modular conditions. The results of this series of experiments are given in Figure 11.

Series 2: Determination of coefficients of discharge :

On the same model, a second series of experiments was conducted to determine the coefficients of discharge for various upstream heads ranging from 2.5 feet to 24 feet under modular conditions. In this series the horizontal length of the anicut model was isolated so that coefficients for constant head over the horizontal length of anicut could be obtained. The results of these experiments are given in Figure 12. The values of h_a were calculated by the formula $Q=L\ (P+H)$ in which P was taken as 10 feet.

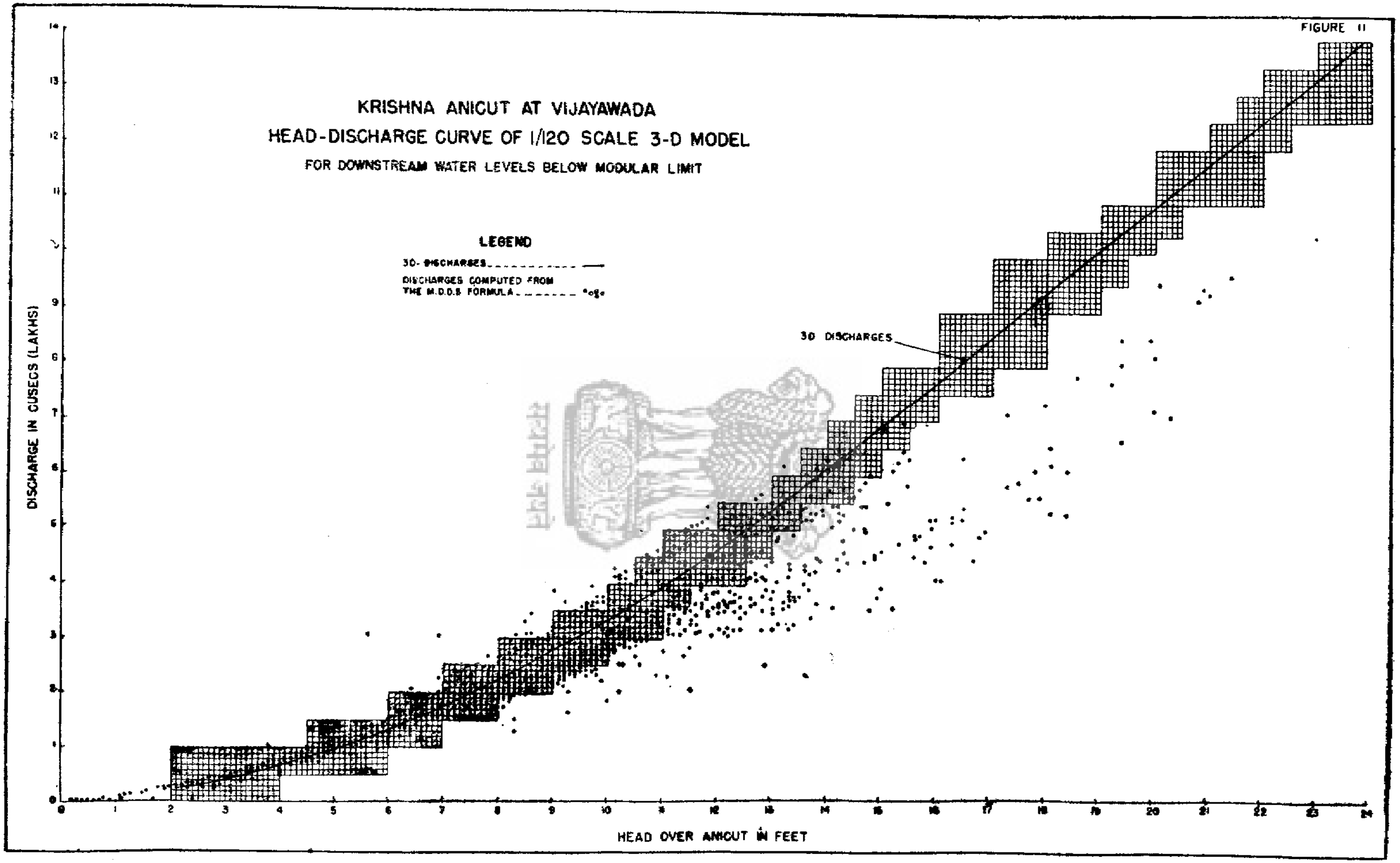


FIGURE 11

KRISHNA ANICUT AT VIJAYAWADA
HEAD-DISCHARGE CURVE OF 1/120 SCALE 3-D MODEL
FOR DOWNSTREAM WATER LEVELS BELOW MODULAR LIMIT

FIGURE 12

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/120 SCALE 3D MODEL

FOR DOWNSTREAM WATER LEVELS BELOW MODULAR LIMIT

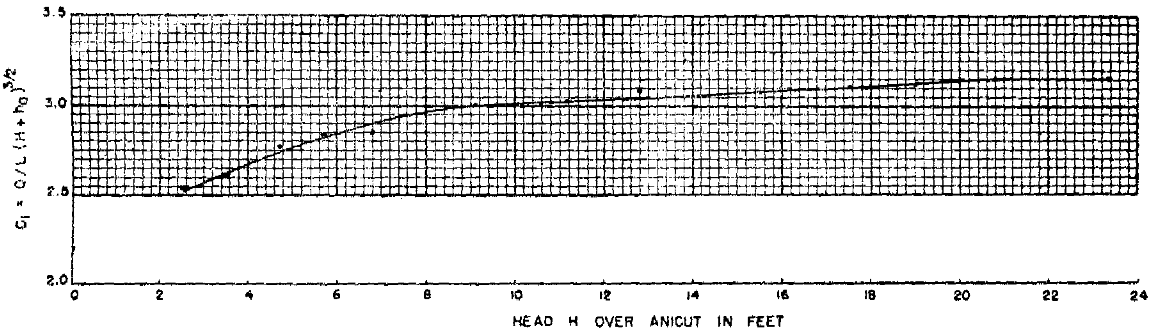


FIGURE 13

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/48 SCALE MODEL

FOR DOWNSTREAM WATER LEVELS BELOW MODULAR LIMIT

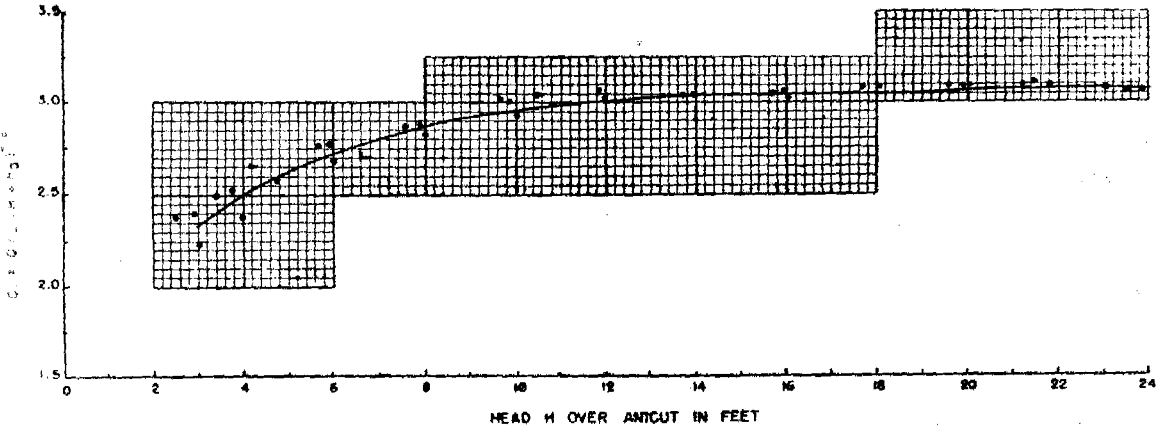


FIGURE 14(b)

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/48 SCALE MODEL

UPSTREAM RIVER BED 30 FEET BELOW CREST OF ANICUT

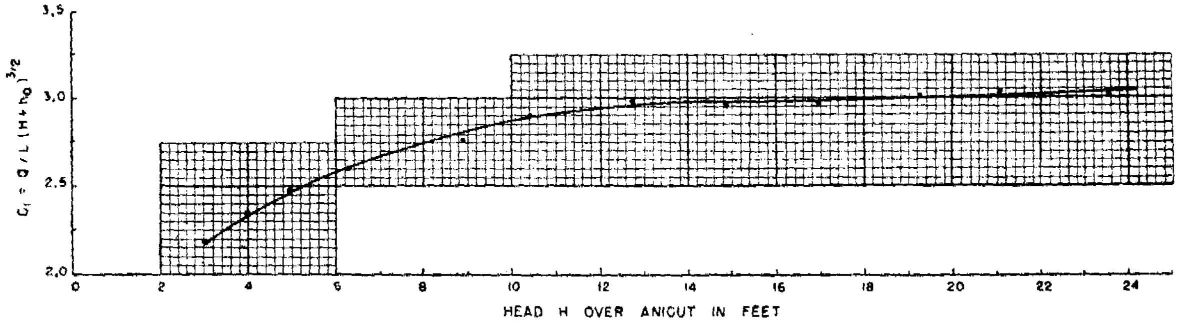


FIGURE 14(a)

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/48 SCALE MODEL

UPSTREAM RIVER BED 50 FEET BELOW CREST OF ANICUT

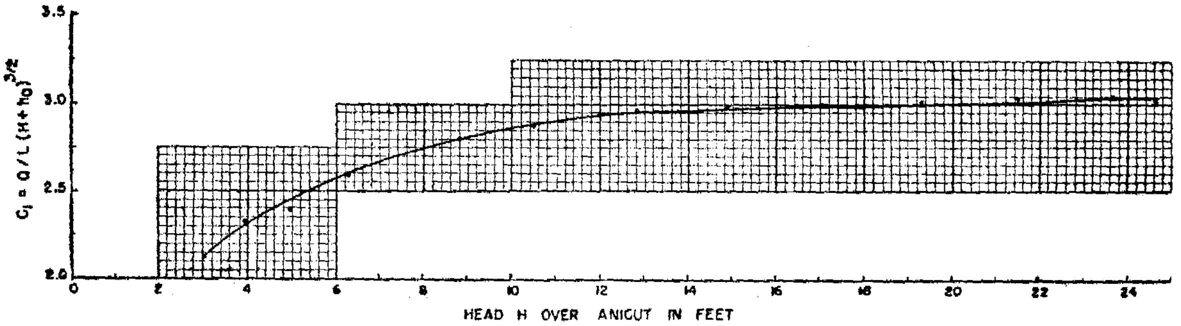


FIGURE 14(c)

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/48 SCALE MODEL

UPSTREAM RIVER BED 20 FEET BELOW CREST OF ANICUT

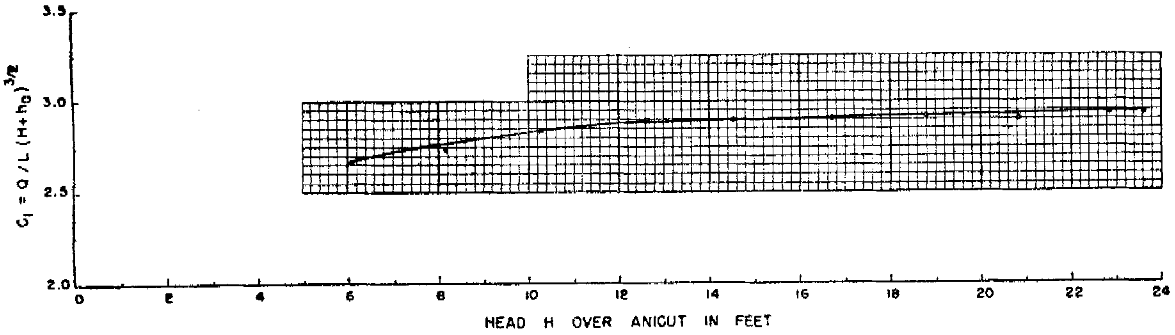


FIGURE 14 (d)

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/48 SCALE MODEL

UPSTREAM RIVER BED 10 FEET BELOW CREST OF ANICUT

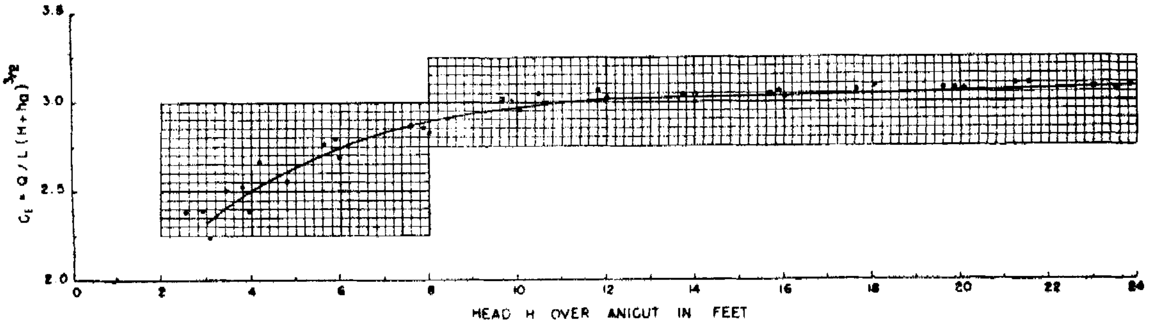
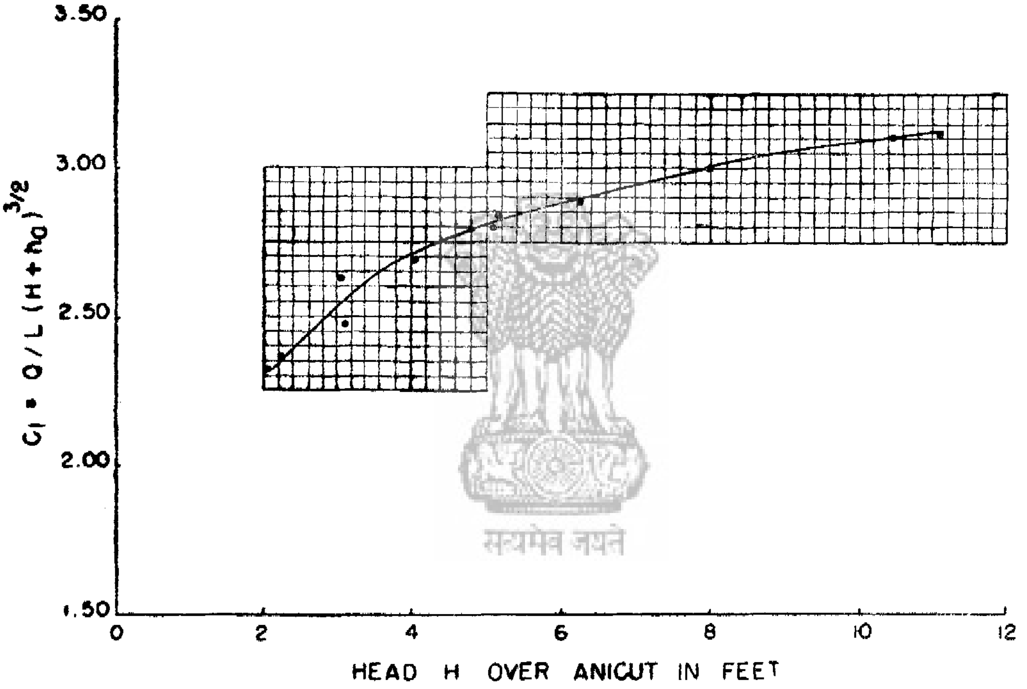


FIGURE 15

KRISHNA ANICUT AT VIJAYAWADA

C_1-H CURVE OF 1/24 SCALE MODEL

FOR DOWNSTREAM WATER LEVELS BELOW MODULAR LIMIT



3. *Studies on a 1/48 scale geometrically similar sectional model:*

Series 1: Coefficient of discharges for various heads with a constant upstream depth of 10 feet below the crest:

Experiments were conducted on a 1/48 geometrically similar sectional model of the anicut in a 3 feet wide flume for determination of the coefficients of discharge for conditions below the modular limit for an upstream depth of 10 feet below the crest of the anicut. The surface of the model was rendered smooth in neat cement. Results of these experiments are given in Figure 13.

Series 2: Coefficients of discharge for different upstream depths below the crest of the anicut:

On the same 1/48 sectional model experiments were conducted to determine the effect of various upstream depths on the coefficient of discharge. These were repeated for upstream depths of 20, 30 and 50 feet below the crest. The results of these as well as those for the 10 feet depth in series 1 are given in Figure 14(a) to 14(d).

4. *Studies on 1/24 scale geometrically similar sectional model:*

Experiments were conducted to determine the coefficients of discharge for the lower heads *i.e.*, 2 feet to 10 feet on a 1/24 scale geometrically similar sectional model of the anicut. These were done with a view to obviate any possible corrections that may be necessary for the lower heads in view of the smallness of the previous model. These experiments were conducted for an upstream depth of 10 feet below the crest of the anicut. The results of these are given in Figure 15.

PART V

GODAVARI ANICUT AT DOWLAISHWARAM

EXPERIMENTS BY THE CENTRAL WATER AND POWER RESEARCH STATION POONA[*]

1. *Introduction:*

Chairman, Krishna Godavari Commission requested the studies to be carried out to find the modular range and coefficient of discharge both for the weir working within and outside the modular range for Godavari Anicut at Dowlaishwaram. The anicut is divided into four different sections namely Dowlaishwaram, Ralli, Maddur and Vizeshwaram sections divided by islands *vide* Plate II.

2. *Terms of reference:*

The Chairman, Krishna Godavari Commission in his D. O. Letter No. KGC-11-61 dated 5th Jan. '62, enunciated the problem in the following terms. It was agreed that all these studies would be restricted to two dimensional sectional models of the different sections of the Godavari Anicut in a flume.

(i) Determine by model experiments, the modular range for each of the four sections of the Anicut at Dowlaishwaram for all the values of head on the crest up to 18 feet. In other words ascertain for each value of H (2 feet, 4 feet and 6 feet up to 18 feet) the highest downstream water level at which the flow over the anicut is not affected in any way by the downstream water level.

(ii) Within the modular range and for different values of H determine (a) the value of coefficient C in the formula $q=CH^{3/2}$ (b) the values of the coefficient C_1 in the formula $q_1=C_1(H+h_a)^{3/2}$.

(iii) In the non-modular range determine the values of the coefficient C_1 in the formula $q=C_1(H+h_a)^{3/2}$ for different values of H and different drowning ratios.

(iv) In determining the coefficients of discharge C and C_1 it may be stated whether there is any scale effect and whether there is any effect of the level of upstream bed on the value of coefficient of discharge.

(v) Verify by model experiments the accuracy of the following formula

$$Q=3\cdot 1L[(h+h_a)^{3/2}-h_a^{3/2}+C_2.L.d\sqrt{2g(h+h_a)}]$$

3. *Experimental Set-up:*

1/40 scale geometrically similar sectional models of all the four sections of the Godavari Anicut were constructed in a glass sided flume four feet wide. All the details of the weir sections were reproduced as shown in Plate II. In all the models the bed level on the downstream side was laid according to the level at the end of the loose stone apron.

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[*] C.W.P.R.S. Specific Note No. 552 dated 3rd July '62.

To ascertain the effect of the upstream bed level on the value of coefficient of discharge, experiments were conducted in 1/40 geometrically similar models of Dowlaishwaram and Maddur with the upstream bed laid rigid 20 feet below the crest *i.e.*, at R.L. 18.75.

The gauges in all the above models were located according to the standard laboratory practice. The gauge upstream of the anicut was located at a point 72 feet away from the anicut crest which was four times the maximum depth of flow expected over the same.

The downstream gauge was so located as to be free from turbulence below the hydraulic jump. At maximum depth of flow of 18 feet over the anicut, the hydraulic jump was seen to form about 50 feet below the crest. The downstream gauge was located about 10 times the depth of the standing wave below its position *i.e.*, 325 feet below the crest of the anicut.

4. *Experiments*:

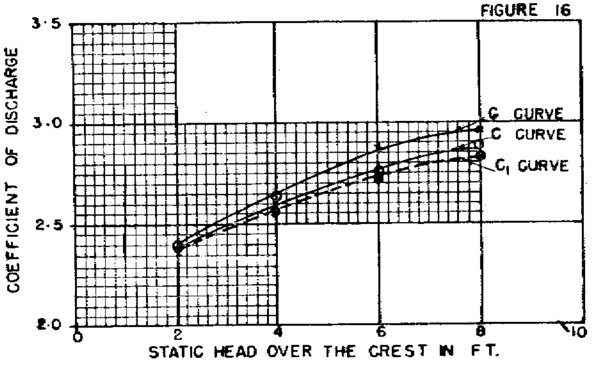
4.1 The experiments were conducted with the static head over the crest as measured at the upstream gauge at intervals of 2 feet from 2 feet to 18 feet. Discharge required was admitted to the model so as to attain the desired water level upstream. The water level on the downstream was maintained so as to form a clear standing wave on the downstream glacis slope. The downstream water level was then gradually raised by a control on the downstream till a limit was reached when any further raising of the downstream water level was found to affect the upstream water level. This gave the limit of modular range. Figures (not printed) show the curves connecting the upstream and the downstream water levels at the limit of modularity as observed in the models. The mean curves connecting the observed upstream water levels with the corresponding observed downstream water levels for different sections of the anicut in the prototype supplied by the Krishna Godavari Commission are also plotted on the same graphs. The coefficients for the ranges within and beyond modular limit have to be evaluated separately.

4.2 *Coefficient of discharge for modular conditions:*

Discharge corresponding to different depths of flow over the weir, within the modular range were measured on the Rehbock weir fitted at the downstream end. The depth of flow at each stage was measured on the upstream gauge. The velocity of approach in the flume was calculated from the observed discharge intensity and the depth of flow. The head due to velocity of approach was calculated from the equation $h_a = \frac{\alpha \times V^2}{2g}$. In this equation α is a coefficient to allow for non-uniform distribution of velocities in the approach channel. This value can be determined experimentally. It increases as the ratio

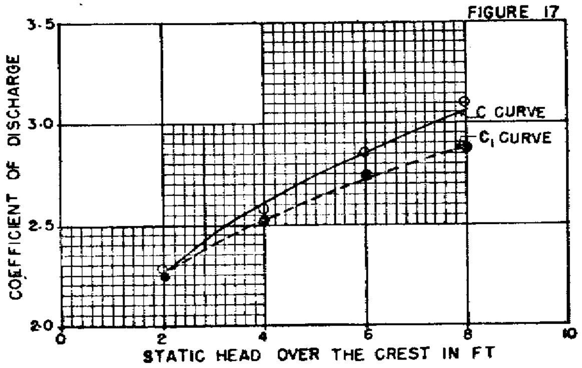
$$\frac{\text{Static Head over the crest } H}{\text{Height of crest above upstream channel bed } P.}$$

FIGURE 16



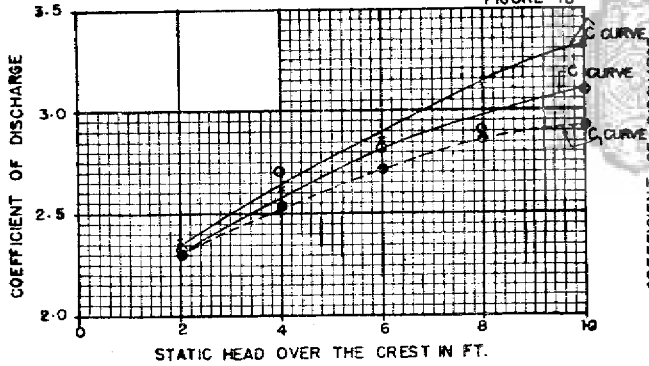
DOWLAISHWARAM SECTION

FIGURE 17



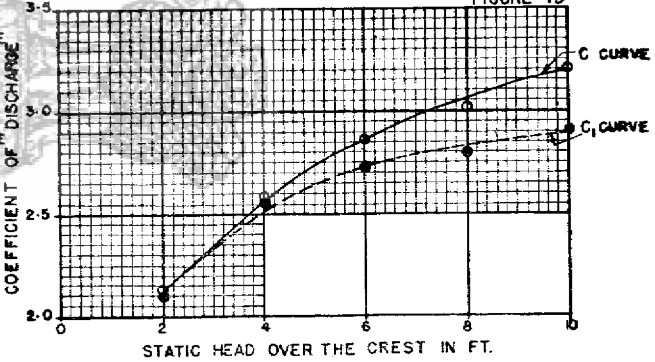
RALLI SECTION

FIGURE 18



MADDUR SECTION

FIGURE 19



VIZESWARAM SECTION

LEGEND

COEFFICIENT FROM THE FORMULA $Q = C H^{3/2}$ WHEN THE U/S BED LAID AS IN PLATE II. ×——×
COEFFICIENT FROM THE FORMULA $Q = C H^{3/2}$ WHEN THE U/S BED IS 20 FT. BELOW CREST. ○——○
COEFFICIENT FROM THE FORMULA $Q = C_1 (H + h_a)^{3/2}$ ●- - -●

GODAVARI ANICUT
GRAPHS SHOWING COEFFICIENT OF DISCHARGE
VERSUS
STATIC HEAD OVER THE CREST
WITHIN MODULAR LIMIT

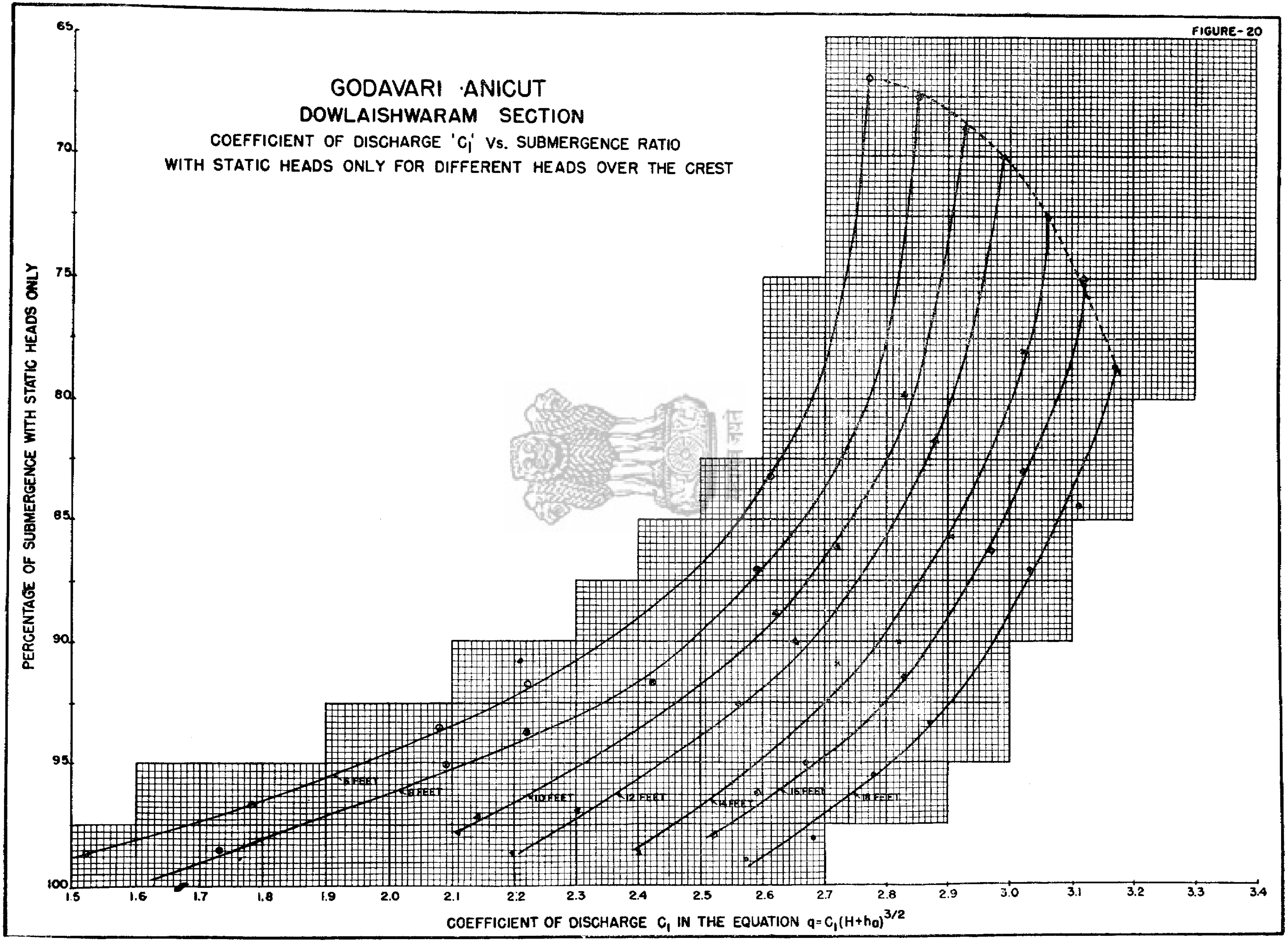


FIGURE-20

GODAVARI ANICUT
DOWLAISHWARAM SECTION
COEFFICIENT OF DISCHARGE 'C_1' Vs. SUBMERGENCE RATIO
WITH STATIC HEADS ONLY FOR DIFFERENT HEADS OVER THE CREST

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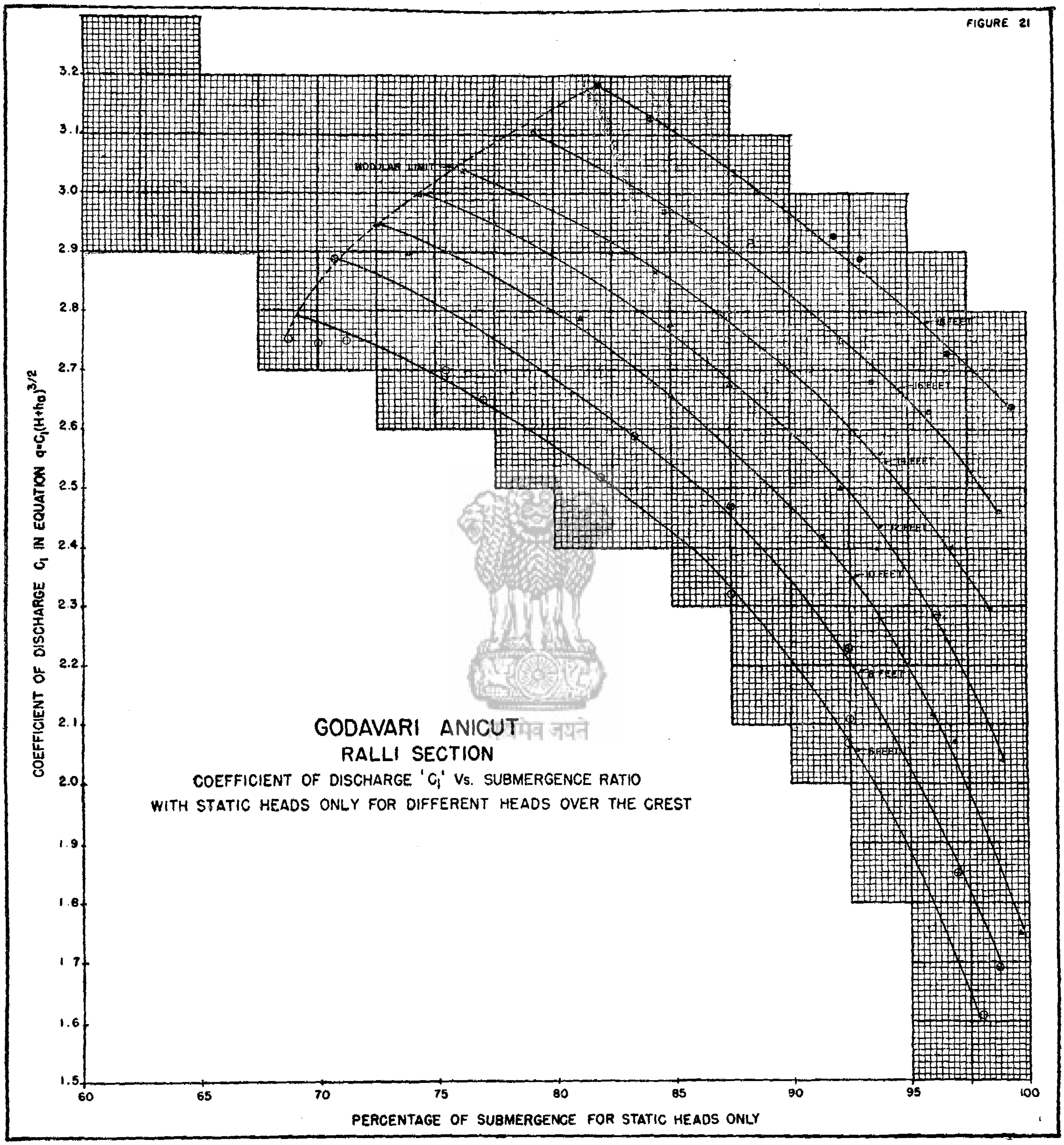
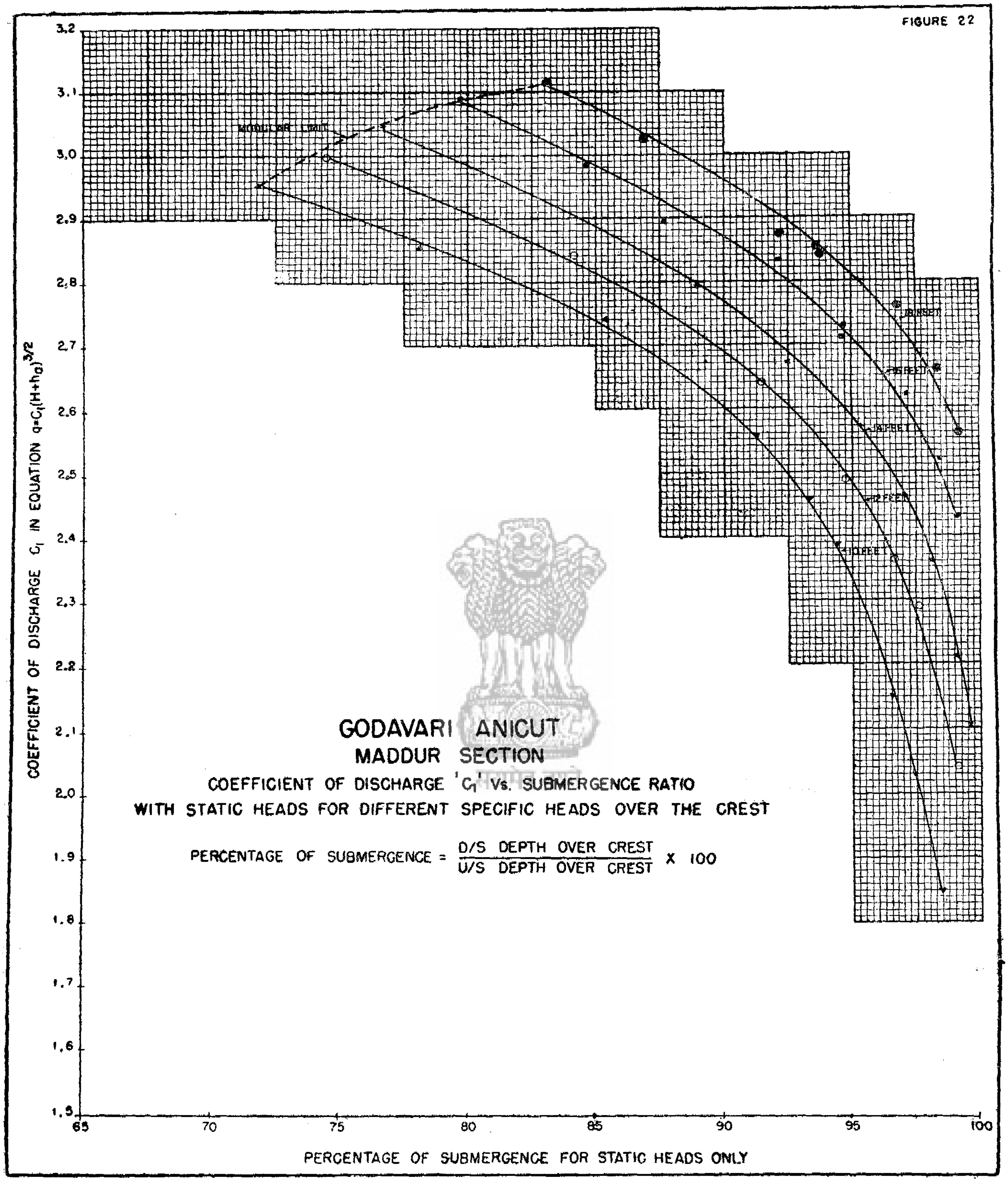


GODAVARI ANICUT
RALLI SECTION
COEFFICIENT OF DISCHARGE 'C_1' Vs. SUBMERGENCE RATIO
WITH STATIC HEADS ONLY FOR DIFFERENT HEADS OVER THE CREST

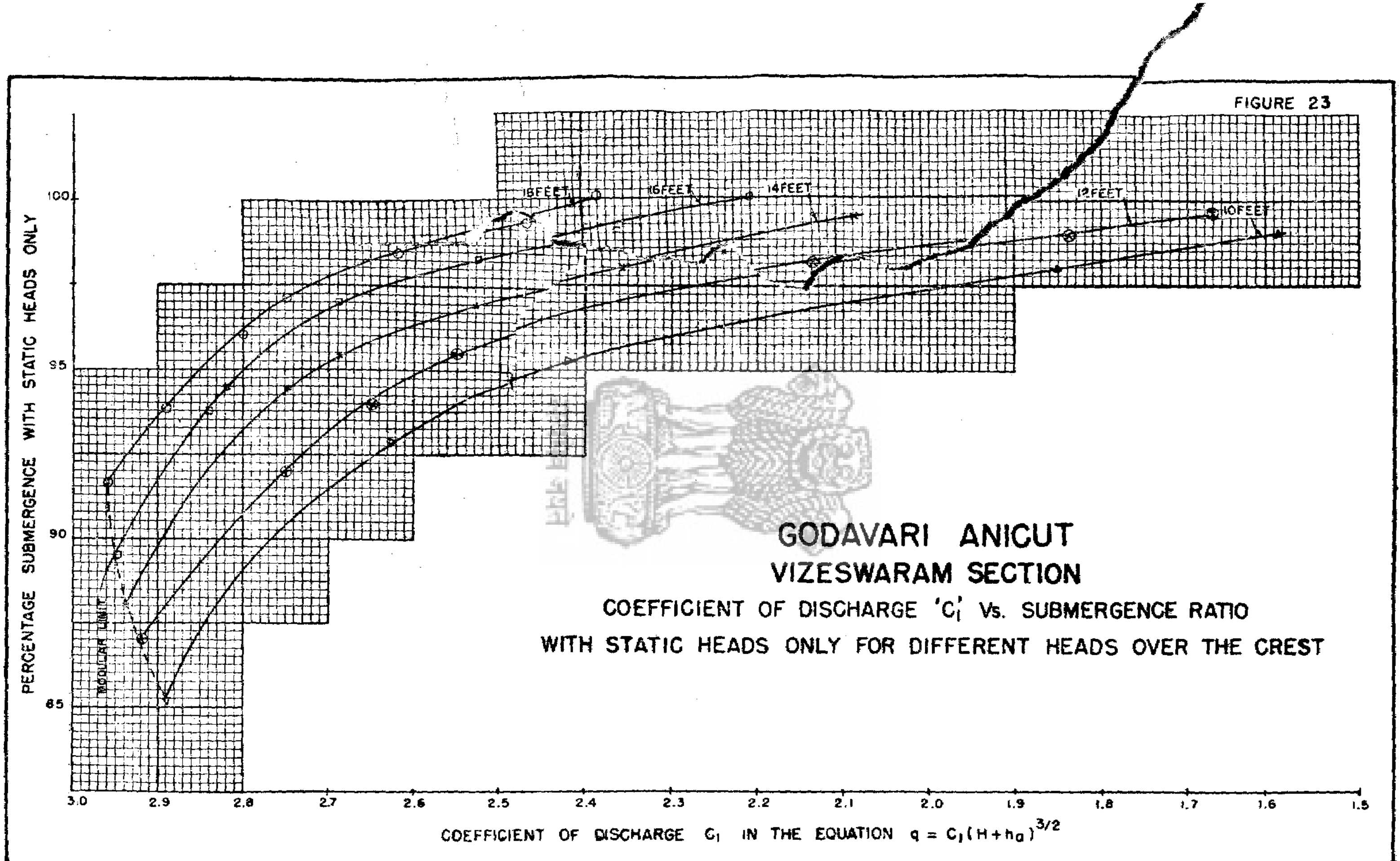
FIGURE 22



GODAVARI ANICUT
MADDUR SECTION
COEFFICIENT OF DISCHARGE 'C_1' Vs. SUBMERGENCE RATIO
WITH STATIC HEADS FOR DIFFERENT SPECIFIC HEADS OVER THE CREST

$$\text{PERCENTAGE OF SUBMERGENCE} = \frac{\text{D/S DEPTH OVER CREST}}{\text{U/S DEPTH OVER CREST}} \times 100$$

FIGURE 23



GODAVARI ANICUT

VIZESWARAM SECTION

COEFFICIENT OF DISCHARGE 'C_1' Vs. SUBMERGENCE RATIO

WITH STATIC HEADS ONLY FOR DIFFERENT HEADS OVER THE CREST

decreases. Its value varies from 1.024 to 1.1 according to the variation of the ratio of head over the crest to the height of weir *i.e.*, H/P. The total energy head upstream was then computed. The coefficients of discharge C and C_1, in the equations $q=CH^{3/2}$ and $q=C_1(H+h_a)^{3/2}$ were then computed. The figures 16 to 19 show the values obtained in the experiments for the four different sections of the anicut.

4.3 *Coefficient of discharge for non-modular conditions*:

In the case of non-modular flow, the discharge varies not only acording to the upstream water level but also according to the downstream water level The upstream water level was, therefore, held constant for various values beyond the modular limit at 2 feet intervals. For each given upstream water level the down stream water level was controlled at a value higher than the corresponding modular limit and the discharge required to maintain upstream water level at the desired value was measured over the Rehbock weir. The head due to V_a was computed as described in the previous paragraph. The coefficient C_1 in the equation $q_1=C_1(H+h_a)^{3/2}$ was then computed and a family of curves showing relation between C and the submergence ratio plotted for different values of heads over the crest, *vide* Figures 20 to 23.

5. *Discussions of results* :

5.1 *Limit of modularity*

Figures (not printed) show the graphs connecting the upstream and downstream water levels at the limit of modularity for various sections of the anicut. Also on these graphs the mean curves connecting the upstream water level with the corresponding observed downstream water levels for different sections in the prototype are plotted. It will be seen from these modularity curves that each section of the anicut has different range of modular working. In Figures (not printed) for Dowlaishwaram and Maddur sections, the modular limit curves for the sections when the upstream bed is 20 feet below the crest are also plotted. It will be seen that the modular limit curves when the upstream bed was at R.L. 18.75 give a comparatively lower limit throughout the range. The lowering of the upstream bed has an effect of reducing the total energy upstream. The discharge intensity corresponding to a fixed value of upstream water level would, therefore, decrease. This results in lowering of the downstream water level at the limit of modularity.

5.2 *Coefficient of discharge within the modular range*

The coefficient C_1 in the equation $q_1=C_1\,(H+h_a)^{3/2}$ as computed for various sections when working within the modular limit have been plotted in figures 16 to 19. Each section has different coefficients at the various upstream water levels over the crest as each section is independent in its working. These curves broadly indicate that the coefficient C_1 is increasing with the increase in head over the crest.

The values of coefficient C in the formula $q=CH^{3/2}$ also have been computed for four sections and are plotted on the same curves. On the Figures 12 and 14 for Dowlaishwaram and Maddur sections, the values of C when the upstream bed is 20 feet below the crest are also plotted. A study of these curves indicate that for the same upstream water level, the value of coefficient C is lower, when the upstream bed is lower. This is explained by the fact that for a fixed value of the upstream static head, there is a reduction in the approach velocity resulting from the lowering of the upstream bed level. This reduction of V_a naturally reduces the corresponding head due to velocity of approach and hence the resultant total energy upstream of the weir and the intensity of discharge. Hence it is clear that the lower the upstream bed, the lower the discharge intensity though the coefficient C_1 in the formula $q_1=C_1\ (H+h_a)^{3/2}$ will remain practically the same because the range of change of total energy is small.

5·3 *Coefficient of discharge in the non-modular range*

As indicated earlier, in the non-modular range the coefficient of discharge varies not only according to upstream water level but also according to downstream water level. No generalisation for the coefficients for various upstream water levels over the crest alone is possible. As the coefficient of discharge varies with the submergence ratio, there can be a number of values of C_1 for the same upstream water level depending upon the downstream water level and also there can exist a number of pairs of upstream and downstream water levels with the discharge remaining constant.

Figures 20 to 23 have been plotted from the observations made as explained in the paragraph 4.3 for various sections of the weir. For each section of the anicut, a family of curves for different heads over the crest at 2 feet intervals have been plotted. Also in a similar way a family of curves for the sections Dowlaishwaram and Maddur when the upstream bed is at R. L. 18.75 have been given in Figures (not printed). Here too, with the studies so far conducted, no generalisation with regard to the effect of upstream bed over the value of C_1 has been possible as the number of variables in the non-modular flow effecting the value C_1 are many. So separate sets of curves for each case are to be drawn.

In all the above cases, the coefficients have been computed with the V_a taken into consideration and the formula used being $q_1=C_1\ (H+h_a)^{3/2}$. It is found not possible to give curves with coefficient C computed from the formula $q=CH^{3/2}$ where the effect of V_a is to be merged with the value of C. From the experiments conducted it was observed that for the same static head over the crest there was a large variation in the range of intensity of discharge when measured at the modular limit and at a fully submerged conditions say at 98 per cent. This gave different values of V_a, for different downstream water levels even though the static head upstream remained constant as there was a change in the intensity of discharge. Hence it is not possible to merge the effect of V_a into the value of coefficient C as each set of upstream and downstream water levels will be having different values of velocity of approach.

5·4 *Verification of the formula*

When the anicut secton is working within the modular range, it is seen that coefficient of discharge C_1 does not depend on the downstream water level. In the case of the formula $Q=3.1\ L[(h+h_a)^{3/2}-h_a^{\ 3/2}]+C_2\ L.d\sqrt{2g\,(h+h_a)}$, the symbol has altogether a different meaning, it being equal to the difference between upstream and downstream water levels. Hence according to this formula, within the modular limit, *i.e.*, when the downstream water levels are lower than those at the modular limit, the values of coefficients will change along with the downstream water level with the same discharge intensity. Alternatively if the values of C_2 as given in the Annexure V are taken, for a particular head there will be a considerable change in the discharge intensities by the above formula for different values of downstream water levels though the discharge intensity for any downstream water level below the modular limit is constant. Some calculations are made for Dowlaishwaram section when working within the modular range, for $H=6$ feet and are shown in *Appendix 2.*

When the anicut is working in non-modular range, some calculations are made for the Ralli section of the anicut by substituting various values as per model results and from Annexure V in the formula $Q=3.1L[(h+h_a)^{3/2}-h_a^{3/2}]+C_2L.d\sqrt{2g(h+h_a)}$. The variations in the values of discharge intensities got from the calculations as against those observed in the model can be seen from this *vide Appendix 3.*

5·5 *Scale effect*

No separate studies for studying the scale effect were made in the case of Godavari Anicuts. However, these studies were made in the case of Krishna Anicut where two sectional models to different scales of 1/40 and 1/25 were constructed. The findings of these studies have been already reported in Part II of this Annexure. An important extract is given below :—

> " Results of models constructed to 1/40 scale (a) and 1/25 scale (b) indicate that the values of coefficient C_1 as obtained for static heads between 8 feet and 24 feet are practically the same. However, below a static head of 8 feet the coefficient obtained from 1/25 scale model are slightly lower than those obtained from 1/40 scale model, this difference being 4% to 5% corresponding to static heads of 4 feet and 2 feet respectively. The difference corresponding to head of 6 feet is somewhat less,"

The same findings and explanation will apply to the Godavari Anicuts also.

6. *Calculation of Discharge :*

6·1 *Within the modular range*

Various curves giving values of the discharge coefficients for various sections have been given in the Figures 16 to 19. The value of the coefficient of discharge actually varies according to the total energy upstream of the weir. It would, therefore, be ideal to use C_1 curve computed and plotted from the formula $q_1=C_1\,(H+h_a)^{3/2}$. For computing the discharge flowing over the weir, it would be necessary to estimate the value of the head due to velocity of approach. This can be estimated by actual measurement of velocities in the prototype. Alternatively if instead of velocities the depth of flow upstream of the weir is measured, the actual total energy upstream of the weir can be computed by successive approximations. The C_1 curve is applicable for all depths of flow upstream as total energy uspstream is taken into consideration.

The other alternative would be to use the C_1 curve, in which coefficient C is plotted against static head upstream of the weir. These curves have a disadvantage that a separate curve is required for each level of the upstream bed *i.e.*, depth below the weir crest. Only in the case of Dowlaishwaram and Maddur sections, two curves for different bed levels are given. As such curves would not be available for different depths, the coefficient C_1 will be a matter of approximation only for intermediate depths. Even for making such an approximation, it would be necessary to evaluate the bed level upstream by measurement of depth of flow in the prototype. Once such a depth is measured, it would be better to use a more exact curve *i.e.*, C_1 curve.

6.2 *Non-modular range*

Separate sets of curves for coefficient C_1 for different heads over the crest for various sections of the anicut are given in the Figures 20 to 23. If coefficient is required for intermediate water levels it should be done by interpolation. For correct calculation of the discharge it is necessary to know the head due to V_a at that particular stage.

7. *Limitation of the present note*

The river plan shows that the river downstream of Dowlaishwaram is having a large expansion in plan and also a number of islands have developed. This results in a considerable change in the approach conditions upstream of these weirs. It is of utmost importance to study this aspect of obliquity of approach which has a considerable effect on the value of coefficient of discharge. It will, therefore, be necessary to carry out these tests on a three dimensional model for correct assessment of the discharge coefficients and the modular range of the weir. The present studies have been carried out on geometrically similar sectional models as desired by the Krishna Godavari Commission. In view of this limitation the application of the coefficients computed from the results of the sectional models may not be valid for estimating discharges of the anicut as a whole.

8. *Conclusions*

As a result of the studies so far carried out on the various sections of the Godavari anicut, the following conclusions are drawn.

(i) The modular range for the four sections of the anicut have been determined and are given in Figures (not printed). Modular ranges for the two sections namely Dowlaishwaram and Maddur have also been determined for a condition when the upstream bed level is at R.L. 18.75. These are shown in Figures (not printed).

(ii) The graphs showing values of both the coefficients C and C_1 plotted against the static head for the four sections of the anicut working within the modular range have been given in Figures 16 to 19. In the case of Dowlaishwaram and Maddur sections of the anicut additional experiments were carried out with the upstream bed lowered down to R.L. 18.75 and the coefficients obtained in this condition are shown in Figures 16 and 18.

(iii) Values of the coefficient C_1 obtained under different submergences for particular values of the static head over the crest for the four anicut sections have been plotted in Figures 16 to 19. Figures (not printed) show similar curves of C_1 against submergence ratio for the two anicut sections Dolaishwaram and Maddur with the upstream bed depressed to R. L. 18.75.

(iv)(*a*) No experiments have been carried out for determining the scale effect in case of the Godavari Anicuts. However, results simliar to those obtained for the Krishna Anicut and already reported in Part II of this Annexure are expected to hold valid.

(*b*) As the difference between the crest level of the weir and the upstream bed level increases the velocity of approach reduces thereby reducing the total energy upstream of the weir. This results in decrease in the intensity of discharge flowing over the weir. However, since the range of change of the total energy is very small, the change in the value of the coefficient C_1 also is very small, perhaps negligible.

(v) (*a*) Within the modular range according to the formula $Q=3.1L\,[(h+h_a)^{3/2}-h_a^{3/2}] + C_2\,L.d\,\sqrt{2g\,(h+h_a)}$, the discharge will change according to the downstream water level whereas it is supposed to remain constant. If at all this formula is to be used the values of the coefficients and head due to velocity of approach will have to be properly ascertained. However, according to the Annexure V the values of h_a and C_2 have been fixed and they do not seem to be valid.

(*b*) For working beyond the modular range, h_a has been assumed to be constant, which is rather not correct because it should vary with the discharge intensity. The values of the coefficient C_2 as given in the Annexure V also do not seem to hold good.

Calculations of discharge intensities from the equation

$$Q=3.1L\left\{(h+h_a)^{3/2}-h_a{}^{3/2}\right\}+C_2.L.d\sqrt{2g(h+h_a)}$$

when the weir section is working within the modular limit :

The calculations presented here are for Dowlaishwaram Section of the anicut for an upstream water level of 44.75 *i.e.*, H=6 feet. The following table shows the value of the intensity of discharge q obtained by different methods.

Serial number	*Upstream water level*	*Down stream water level*	*h*	*d*	*C2 as given in the Annexure V*	*q when ha=0·13 as per model*	*q when h=0·145 given in Annexure*	*q obtained in the model*
1	2	3	4	5	6	7	8	9
1	44·75	42·77	1·98	4·02	0·60	37·46	37·65	42·00
2	44·75	41·77	2·98	3·02	0·60	42·46	42·60	,,
3	44·75	40·77	3·98	2·02	0·60	45·43	45·60	,,
4	44·75	39·77	4·98	1·02	0·60	46·60	47·86	,,

The values of q in the columns 7 and 8 firstly indicate the difference in the discharge intensity likely to be caused due to change in the head corresponding to the velocity of approach, both as given in the Annexure V and as found in the model. Another point to note is the great difference that all these calculated intensities are showing from the observed discharge intensity of 42.0 cusecs per foot run.

Appendix 3

Calculations of discharge intensity from the equation

$$Q=3.1L\left\{(h+h_a)^{3/2}-h_a^{3/2}\right\}+C_2L.d\sqrt{2g(h+h_a)}.$$

when the weir is working in the non-modular range :

The calculations reported here are for Ralli Section of the anicut when working in the non-modular range for 10 feet and 6 feet static heads over the crest. The various downstream water levels are the values actually observed in the model.

Different values of the intensities of discharge q obtained by calculating from the present equation with various combinations are presented below. Tables 10(a) and 10 (b) are for 10 feet head over the crest and Tables 11 (a) and 11 (b) are for 6 feet head over the crest. In the Tables 10 (a) and 11 (a) values of intensity of discharge calculated for various observed downstream water levels, using the values of C_2 and V_a of the Annexure V are presented. In the tables 10(b) and 11(b), the values of intensity of discharge calculated for the same observed downstream water levels, using the values of V_a as obtained in the model and the value of C_2 as given in the Annexure V are presented.

Tables 10(c) and 11(c) are drawn from the above tables to which are added the actual observed intensities of discharge in the model at corresponding stages.

The values of q in the columns 6 and 7 of Tables 10(c) and 11(c) firstly indicate the difference in the discharge intensity q likely to be caused due to change in the head corresponding to the velocity of approach, both as given in the Annexure II and as found in the model. It may be seen that in this Annexure for a specific static head over the crest only one value of V_a is assumed irrespective of the submergence(*i.e.*, downstream water levels). Actually this is not true and each pair of downstream and upstream water levels will be having different values of V_a corresponding to change in the intensity of discharge with the submergence ratio. For example, from the Annexure a value of 4.12 feet per second has to be assumed as V_a for all submergence ratios corresponding to static head of 10 feet. But from the model it is found that this value of V_a varies from 6.15 feet per second at the modular limit to 3.37 feet per second at 99.6 per cent submergence.

Another point to note is the difference that all these calculated intensities are showing [columns 6 and 7 of Tables 10(c) and 11 (c)] from the discharge intensity actually observed on the model [column 8 of Tables 10(c) and 11 (c)].

Also there will be a change in the value of discharge intensity due to lowering of the upstream bed.

It will thus be seen that the calculated intensities are varying to a large extent depending upon the value of C_2 and V_a assumed for the calculations.

TABLE No. 10(a)

Ralli Section

Calculations using values of C_2 and V_a as per Annexure V for H=10 feet

Sl. No.	*Upstream water level*	*Down-stream water level*	*h*	*d*	*V_a as per Annexure V*	$h_a=\alpha V_a^2/2g$	*C_2 as per Annexure V*	*q calculated from the equation*	*q as obtained in the model*
1	2	3	4	5	6	7	8	9	10
1	48·75	46·00	2·75	7·25	4·12	0·288	0·66	82·92	102·00
2	,,	46·87	1·88	8·12	,,	,,	0·75	81·50	95·00
3	,,	47·27	1·48	8·52	,,	,,	0·75	74·92	90·00
4	,,	47·91	0·84	9·16	,,	,,	0·80	65·54	80·80
5	,,	48·35	0·40	9·60	,,	,,	0·85	55·79	70·00
6	,,	48·71	0·04	9·96	,,	,,	0·90	42·22	57·00

TABLE No. 10(*b*)

Ralli Section

Calculations using the values of C_2 of the Annexure V for H=10 feet and V_a as obtained in the model

Sl. No.	*Up stream water level*	*Down stream water level*	*h*	*d*	V_a	$h_a=\alpha V_a^2/2g$	*C_2 as per Annexure V*	*q from the formula*	*q from the model*
1	2	3	4	5	6	7	8	9	10
1	48·75	46·00	2·75	7·25	6·15	0·609	0·66	87·92	102·00
2	,,	46·87	1·88	8·12	5·62	0·51	0·75	85·80	95·00
3	,,	47·27	1·48	8·52	5·32	0·46	0·75	78·51	90·00
4	,,	47·91	0·84	9·16	4·78	0·37	0·80	68·12	80·80
5	,,	48·35	0·40	9·60	4·14	0·28	0·85	55·33	70·00
6	,,	48·71	0·04	9·96	3·37	0·18	0·90	33·88	57·00

TABLE No. 10(*c*)

Ralli Section

Discharge intensities calculated from various methods for H = 10 feet

Serial No.	*Upstream water level*	*Downstream water level*	*h*	*d*	*q calculated with V_a & C_2 from Annexure V*	*q calculated with V_a model and C_2 from Annexure V*	*q actually observed in model*
1	2	3	4	5	6	7	8
1	48·75	46·00	2·75	7·25	82·92	87·92	102·00
2	,,	46·87	1·88	8·12	81·50	85·80	95·00
3	,,	47·27	1·48	8·52	74·92	78·51	90·00
4	,,	47·91	0·84	9·16	65·54	68·12	80·80
5	,,	48·35	0·40	9·60	55·79	55·33	70·00
6	,,	48·71	0·04	9·96	42·22	33·88	57·00

TABLE No. 11(*a*)

Ralli Section

Calculations using values of C_2 and V_a as per annexure V for H=6 Feet.

Sl. No.	*Upstream water level*	*Down-Stream water level*	*h*	*d*	*Va as per Annexure V*	$h_a = \propto V_a^2/2g$	*C_2 as per Annexure V*	*q calculated from the equation*	*q observed in the model*
1	2	3	4	5	6	7	8	9	10
1	44·75	42·87	1·88	4·12	3·52	0·207	0·60	37·70	42·00
2	,,	43·37	1·38	4·62	,,	,,	,,	33·92	38·50
3	,,	43·67	1·08	4·92	,,	,,	,,	31·09	38·30
4	,,	43·99	0·76	5·24	,,	,,	,,	27·46	35·20
5	,,	44·31	0·44	5·56	,,	,,	,,	22·84	30·50
6	,,	44·43	0·32	5·68	,,	,,	,,	20·72	28·44
7	,,	44·63	0·12	5·88	,,	,,	,,	16·42	23·73

TABLE No. 11(*b*)

Ralli Section

Calculations using the value of C_2 of Annexure V for H=6 feet and V_a of model.

Sl. No.	*Up-stream water level*	*Down-stream water level*	*h*	*d*	*Va*	*ha*= $\propto V_a^2/2g$	C_2 *as per Annexure V*	*q from the equation*	*q from the model*
1	2	3	4	5	6	7	8	9	10
1	44·75	42·87	1·88	4·12	3·25	0·174	0·60	37·27	42·00
2	,,	43·37	1·38	4·62	3·15	0·163	,,	33·32	40·50
3	,,	43·67	1·08	4·92	2·97	0·145	,,	30·23	38·30
4	,,	43·99	0·76	5·24	2·70	0·12	,,	26·05	35·20
5	,,	44·31	0·44	5·56	2·37	0·09	,,	20·51	30·50
6	,,	44·43	0·32	5·68	2·20	0·079	,,	17·96	28·44
7	,,	44·63	0·12	5·88	1·84	0·06	,,	12·09	23·73

TABLE No. 11(*c*)

Ralli Section

Discharge intensities calculated from various methods for H=6 feet.

Serial No.	*Upstream water level*	*Downstream water level*	*h*	*d*	*q calculated with Va and C_2 from Annexure V*	*q calculated with Va from model and C_2 from Annexure*	*q obtained in the model*
1	2	3	4	5	6	7	8
1	44·75	42·87	1·88	4·12	37·70	37·27	42·00
2	,,	43·37	1·38	4·62	33·92	33·32	40·50
3	,,	43·67	1·08	4·92	31·09	30·23	38·30
4	,,	43·99	0·76	5·24	27·46	26·05	35·20
5	,,	44·31	0·44	5·56	22·84	20·51	30·50
6	,,	44·43	0·32	5·68	20·72	17·96	28·44
7	,,	44·63	0·12	5·88	16·42	12·09	23·73

PART VI

Comparison of calculated and recorded discharges over the Krishna Anicurt at Vijayawada for the days when all the shutters were down

Month and date	*Gauge 6 A.M.*			H=(4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of (6) over (7)*
	Vijayawada	*Seethanagram*	*Average* $=\frac{(2)+(3)}{2}$				
1	2	3	4	5	6	7	8
July, 1947							
21	55·05	55·23	55·14	7·92	220,000	221,051	99·5
22	56·25	56·33	56·29	9·07	283,000	306,794	92·2
23	58·05	58·43	58·24	11·02	398,000	395,371	100·7
24	59·35	59·73	59·54	12·32	486,000	408,815	118·9
25	58·55	59·03	58·79	11.57	434,000	365,371	118·8
26	57·65	58·03	57·84	10·61	370,000	298,499	124·0
29	54·35	54·33	54·34	7·12	182,000	181,942	100·0
31	54·50	54·53	54·52	7·30	190,000	140,393	135·3
August, 1947							
1	54·65	54·63	54·64	7·42	196,000	180,268	108·7
2	54·35	54·53	54·44	7·22	186,000	190,332	97·7
3	54·35	54·53	54·44	7·22	186,000	170,424	109·1
26	57·45	57·83	57·64	10·42	359,000	391,371	91·7
27	59·05	59·33	59·19	11·97	463,000	428,025	108·2
28	58·70	59·03	58·86	11·64	438,000	429,350	102·0
29	59·05	59·23	59·14	11·92	460,000	391,186	117·6
30	57·85	58·13	57·99	10·77	380,000	313,962	121·0
31	57·05	57·23	57·14	9·92	328,000	299,211	109·6

PART VI—*contd.*

Month and date	*Gauge 6 A.M.*			*H*=(4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of* (6) *over* (7)
	Vijayawada	*Seethanagram*	*Average* $\frac{(2)+(3)}{2}$				
1	2	3	4	5	6	7	8
September, 1947							
1	57·00	57·23	57·12	9·90	327,000	297,878	109·8
2	56·95	57·13	57·04	9·82	322,000	298,935	107·7
3	57·35	57·63	57·49	10·27	349,000	332,889	104·8
4	57·55	57·93	57·74	10·52	365,000	339,543	107·05
5	57·65	57·83	57·74	10·52	365,000	360,708	101·2
6	58·25	58·73	58·49	11·27	414,000	365,09	113·4
7	57·75	58·13	57·94	10·72	377,000	311,764	120·9
8	56·95	57·33	57·14	9·92	328,000	279,405	117·4
9	56·65	57·03	56·84	9·62	311,000	262,709	118·4
10	56·50	56·93	56·72	9·50	305,000	257,598	118·4
11	56·65	56·23	56·44	9·22	290,000	233,145	124·4
12	55·35	55·33	55·34	8·12	230,000	207,497	110·8
13	54·55	54·53	54·54	7·32	191,000	183,214	104·2
20	56·65	56·83	56·74	9·52	307,000	259,181	118.5
21	56·85	57·03	56·94	9·72	317,000	340,887	93·0
22	57·15	57·33	57·24	10·02	335,000	260,186	128·8
23	55·75	55·93	55·84	8·62	256,000	235,663	108·6
24	55·90	56·33	56·12	8·90	273,000	258,910	105·4

PART VI—*coutd.*

Month and date	*Gauge 6 A. M.*			*H*=(4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of (6) over (7)*
	Vijayawada	*Seethanagram*	*Average* (2)+(3) / 2				
1	2	3	4	5	6	7	8
25	56·75	57·13	56·94	9.72	317,000	287,021	110·8
26	56·95	57·23	57·09	9·87	326,000	252,666	129·0
27	55·35	55·23	55·29	8·07	228,000	232,690	98·0
28	57·25	57·33	57·29	10·07	337,000	434,590	77·5
29	58·95	59·03	58·99	11·77	449,000	278,861	161·0
30	55·25	55·43	55·34	8·12	230,000	246,907	93·2
October, 1947							
1	57·00	57·23	57·12	9·90	327,000	266,842	122·5
2	55·55	55·63	55·59	8·37	244,000	203,120	120·1
3	54·10	54·33	54·22	7·00	175,000	185,397	94·4
4	54·10	54·13	54·12	6·90	171,000	172,343	99·2
5	53·55	53·63	53·59	6·37	149,000	171,541	86·9
6	53·90	54·03	53·96	6·74	165,000	172,650	95·6
7	53·45	53·43	53·44	6·22	143,000	107,420	133·1
December, 1947							
22	53·35	53·63	53·59	6·37	149,000	6,041	246·6
		TOTAL .	(T.M.C.)		1,264·1	1,141·7	110·7

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)*			*H*=(4)— 47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of* (6) *over* (7)
	Vijayawada	*Seethanagram*	*Average*= (2)+(3) / 2				
1	2	3	4	5	6	7	8
July 1948							
25	54·55	54·73	54·64	7·42	195,000	216,196	90·2
26	55·45	55·93	55·69	8·47	248,000	252,578	78·2
27	56·75	57·23	56·99	9·77	320,000	334,761	95·6
28	58·15	58·73	58·44	11·22	410,000	355,858	115·2
29	57·35	57·63	57·49	10·27	348,000	274,765	126·7
30	56·05	56·43	56·24	9·02	279,000	232,228	120·1
31	55·15	55·43	55·29	8·07	227,000	210,801	107·7
August 1948							
1	54·65	54·83	54·74	7·52	200,000	198,162	100·9
2	54·45	54·63	54·54	7·32	190,000	192,600	98.7
3	54·25	54·23	54·24	7·02	176,000	182,537	96.4
7	54·15	54·13	54·14	6·92	172,000	185,564	92·7
8	54·05	53·93	53·99	6·77	166,000	174,806	95·0
10	54·05	54·33	54·19	6 97	174,000	164,961	105·5
13	54·85	55·13	54·99	7 77	213,000	194,245	109·7
14	54·25	54·33	54·29	7 07	179,000	201,404	88 9
15	55·15	55·63	55·39	8 17	233,000	245,647	94 9
16	56·45	56·93	56·69	9 47	304,000	274,495	110 7
17	56·75	57·23	56·99	9·77	320,000	281,623	113·6
18	56·85	57·33	57·09	9·87	326,000	307,253	106·1
19	57·45	58·03	57·74	10·52	364,000	338,816	107·4
20	57·45	57·93	57·69	10·47	362,000	316,933	114·2

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)*			*H*=(4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of (6) over (7)*
	Vijayawada	*Seethanagram*	*Average*= $\frac{(2)+(3)}{2}$				
1	2	3	4	5	6	7	8
21	57·45	57·93	57·69	10·47	362,000	329,016	110·0
22	57·65	58·13	57·89	10·67	374,000	353,984	105·7
23	58·15	58·73	58·44	11·22	410,000	350,457	117·0
24	58·15	58·63	58·39	11·17	407,000	337,299	120·7
25	57·55	57·93	57·74	10·52	364,000	310,413	117·3
26	56·75	57·03	56·89	9·67	314,000	244,777	128·3
27	55·75	55·73	55·74	8·52	252,000	208,272	121·0
28	54·65	54·73	54·69	7·47	197,000	183,705	107·2
29	54·05	54·03	54·04	6·82	168,000	176,271	95·3
September 1948							
25	57·05	57·43	57·24	10·02	333,000	272,728	122·1
26	56·45	56·63	56·54	9·32	295,000	244,084	120·9
27	55·85	56·23	56·04	8·82	270,000	239,169	112·9
28	55·55	55·83	55·69	8·47	248,000	214,299	115·7
29	54·75	54·93	54·84	7·62	205,000	195,331	105·0
30	54·45	54·63	54·54	7·32	190,000	191,849	99·0
October 1948							
1	54·45	54·53	54·49	7·27	188,000	192,965	97·4
2	54·55	54·63	54·59	7·37	193,000	191,782	100·6
3	54·45	54·73	54·59	7·37	193,000	225,202	85·7
4	55·85	56·13	55·99	8·77	266,000	213,336	124·7
5	54·45	54·53	54·49	7·27	188,000	168,047	111·9
6	53·45	52·93	53·19	5·97	131,000	131,440	99·7

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)* *Vijayawada*	*Seethanagram*	*Average*= $\frac{(2)+(3)}{2}$	*H*= (4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of* (6) *over* (7)
1	2	3	4	5	6	7	8
November 1948							
27	53·25	53·23	53·24	6·02	134,000	145,128	92·3
30	54·25	54·63	54·44	7·22	186,000	174,236	106·8
		Total (T.M.C.)		.	974·1	901·2	108·1
July 1949							
20	56·45	57·03	56·74	9·52	307,000	290,168	105·8
21	57·25	57·83	57·54	10·32	350,000	323,273	108·3
22	57·45	57·93	57·69	10·47	362,000	315,370	114·8
23	56·95	57·33	57·14	9·92	329,000	274,586	119·8
24	56·35	56·73	56·54	9·32	296,000	244,102	121·3
25	55·55	55·83	55·69	8·47	248,000	211,609	117·2
26	54·45	54·73	54·59	7·37	192,000	181,792	105·6
August 1949							
2	54·45	54·83	54·64	7·42	195,000	195,330	99·8
3	54·25	54·43	54·34	7·12	181,000	169,639	106·7
7	55·05	55·43	55·24	8·02	226,000	223,691	101·0
8	55·65	56·13	55·89	8·67	260,000	288,174	90·2
9	58·35	58·93	58·64	11·42	424,000	367,286	115·4
10	58·55	58·93	58·74	11·52	430,000	338,677	127·0
11	57·25	57·73	57·49	10·27	348,000	277,583	125·4
12	56·25	56·53	56·39	9·17	287,000	251,733	114·0
13	56·25	56·53	56·39	9·17	287,000	228,376	125·7
14	54·85	55·13	54·99	7·77	213,000	196,263	108·5

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)*			*H*= (4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of (6) ove (7)*
	Vijayawada	*Seethanagram*	*Average*= $\frac{(2)+(3)}{2}$				
1	2	3	4	5	6	7	8
15	54·25	54·23	54·24	7·02	176,000	180,664	97·4
16	54·25	54·43	54·34	7·12	181,000	179,530	100·8
17	53·95	54·03	53·99	6·77	166,000	175,230	94·7
18	53·85	53·93	53·89	6·67	162,000	174,274	93·0
August 1949							
19	54.05	54·13	54·09	6·87	170,000	180,798	94·0
20	54·05	54·03	54·04	6·82	167,000	156,105	107·0
21	53·65	53·53	53·59	6·37	148,000	167,270	88·5
September 1949							
5	54·65	54·93	54·79	7·57	202,000	208,006	97·1
6	54·85	55·03	54·94	7·72	213,000	191,435	111·3
7	53·75	53·83	53·79	6·57	157,000	145,050	108·2
October 1949							
21	54·05	54·33	54·19	6·97	174,000	150,406	115·7
23	53·95	54·23	54·09	6·87	170,000	192,577	88·3
24	54·95	55·13	55·04	7·82	215,000	201,157	106·9
25	54·15	54·33	54·24	7·02	176,000	168,069	104·7
November 1949							
2	51.45	51·83	51·64	4·42	82,000	75,317	108·9
3	50.45	50·83	50·64	3·42	52,000	50,315	103·3
4	49.85	49·73	49·79	2·57	35,000	38,324	91·3
5	49·55	49·63	49·59	2·37	29,000	24,695	117·4
			TOTAL (T.M.C.)		657·5	608·0	108·1

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)*			H=(4)—47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of (6) over (7)*
	Vijayawada	*Seethanagram*	*Average*= (2)+(3) / 2				
1	2	3	4	5	6	7	8
July 1950							
20	56·45	56·83	56·64	9·42	300,000	277,742	108·0
21	57·25	57·33	57·29	10·07	337,000	306,987	109·8
22	57·95	58·23	58·09	10·87	388,000	380,441	102·0
23	58·25	58·63	58·44	11·22	410,000	365,292	112·2
24	57·15	57·53	57·34	10·12	340,000	303,030	112·2
25	57·85	58·13	57·99	10·77	380,000	403,135	94·3
26	59·75	60·33	60·04	12·82	522,000	511,581	102·0
27	61·95	62·73	62·34	15·12	700,000	557,168	125·6
28	62·90	63·63	63·26	16·04	776,000	545,941	142·1
29	62·45	63·13	62·79	15·57	736,000	493,724	149·1
30	60·75	61·33	61·04	13·82	600,000	472,372	127·0
31	60·20	60·73	60·46	13·24	554,000	482,151	114·9
August 1950							
1	61·30	62·03	61·66	14·44	647,000	520,425	124·3
2	62·55	63·23	62·89	15·67	745,000	513,820	145·0
3	62·50	63·13	62·82	15·60	739,000	477,538	154·8
4	60·45	60·93	60·69	13·47	571,000	404,389	141·2
5	58·05	58·43	58·24	11·02	398,000	306,817	129·7
6	56·35	56·63	56·49	9·27	293,000	228,775	128·1
7	54·95	54·93	54·94	7·72	211,000	189,265	111·5
8	53·85	53·83	53·84	6·62	159,000	148,158	107·3

PART VI—*contd.*

Month and date	*Gauge* (**6** *A. M.*)			*H*=(4)—47·22	*Calculated discharge from curve*)	*Recorded discharge*	*Percentage of* (6) *over* (7)
	Vijayawada	*Seethanagram*	*Average*= (2)+(3) / 2				
1	2	3	4	5	6	7	8
August 1950							
12	54·35	54·43	54·39	7·17	184,000	217,479	84·6
13	55·95	56·23	56·09	8·87	272,000	239,230	113·7
14	56·05	56·13	56·09	8·87	272,000	244,361	111·3
15	56·25	56·43	59·34	9·12	285,000	245,186	116·2
16	56·25	56·53	56·39	9·17	288,000	240,632	119·7
17	55·75	55·93	55·84	8·62	256,000	221,896	115·4
18	54·95	55·13	55·04	7·82	215,000	192,744	111·5
September 1950							
12	61·85	62·23	62·04	14·82	678,000	534,128	126·9
13	62·15	62·93	62·54	15·32	717,000	552,218	129·8
14	62·95	63·23	63·09	15·87	763,000	463,365	164·7
15	58·65	58·93	58·79	11·57	434,000	329,767	131·6
16	56·85	57·03	56·94	9·72	317,000	260,695	121·6
17	56·25	56·53	56·39	9·17	288,000	252,419	114·1
18	56·65	56·93	56·79	9·57	310,000	264,166	117·4
19	56·35	56·43	56·39	9·17	288,000	230,202	125·1
20	55·05	55·23	55·14	7·92	219,000	190,516	115·0
October 1950							
12	54·25	54·43	54·34	7·12	181,000	211,205	85·7
13	55·55	55·83	55·69	8·47	248,000	227,655	108·9
14	55·15	55·33	55·24	8·02	225,000	203,340	110·7
15	54·15	54·13	54·14	6·92	172,000	189,776	90·6

PART VI—*contd.*

Month and date	*Gauge (6 A.M.)*			H= (4)— 47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of* (6) *over* (7)
	Vijayawada	*Seetha-nagram*	*Average*= (2)+(3) / 2				
1	2	3	4	5	6	7	8
18	54·65	54·93	54·79	7·57	203,000	217,990	93·1
19	55·25	55·53	55·39	8·17	233,000	216,405	107·7
20	54·55	54·83	54·69	7·47	198,000	177,271	111·7
22	53·05	53·63	53·34	6·12	138,000	117,584	117·4
			TOTAL (T.M.C.)		1,485·2	1,220·7	121.7
July 1950							
25	54·25	54·53	54·39	7·17	184,000	207,433	88·7
26	55·35	55·63	55·49	8·27	237,000	214,466	110·5
27	54·35	54·73	54·54	7·32	190,000	205,736	92·4
28	55·65	56·03	55·84	8·62	257,000	262,845	97·8
29	56·75	57·13	56·94	9·72	317,000	285,181	111·2
30	57·05	57·53	57·29	10·07	337,000	318,739	105·7
31	57·55	58·13	57·84	10·62	370,000	351,450	105·3
August 1950							
1	57·75	58·13	57·94	10·72	377,000	315,597	119·5
2	57·05	57·23	57·14	9·92	329,000	287,316	114·5
3	57·35	57·93	57·64	10·42	358,000	306,616	116·8
4	56·65	56·93	56·79	9·57	310,000	249,627	124·2
5	55·85	56·13	55·99	8·77	266,000	231,973	114·7
6	55·55	55·83	55·69	8·47	248,000	229,809	107·9
7	55·75	56·13	55·94	8·72	263,000	233,167	112·8
8	55·55	55·73	55·64	8·42	246,000	213,869	115·0

PART VI—*contd.*

Month and date	*Gauge (6 A. M.)*			*H*= (4)— 47·22	*Calculated discharge (from curve)*	*Recorded discharge*	*Percentage of* (6) *over* (7)
	Vijayawada	*Seethanagram*	*Average*= $\frac{(2)+(3)}{2}$				
1	2	3	4	5	6	7	8
9	54·75	55·03	54·89	7·67	209,000	197,357	105·9
12	54·65	54·83	54·74	7·52	200,000	199,727	100·1
13	55·25	55·53	55·39	8·17	233,000	267,404	87·1
August 1951							
14	57·45	57·93	57·69	10·47	362,000	337,931	107·1
15	57·55	58·03	57·79	10·57	367,000	297,478	123·4
16	56·65	57·03	56·84	9·62	312,000	249,254	125·2
17	55·75	56·03	55·89	8·67	260,000	218,448	119·0
18	54·75	54·93	54·84	7·62	205,000	177,390	115·6
October 1950							
7	55·25	55·53	55·39	8·17	233,000	222,154	104·9
			TOTAL (T.M.C.)		576·3	525·4	109.7

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